VIEW OF CITY OF PANAMA FROM ANCON HILL

In the foreground is shown the new concrete reservoir and at the left the Canal Administration Building in process of erection.

Courtesy of Panama Canal Commission, United States Government, Washington, D C.

Cyclopedia
of
Civil Engineering

A General Reference Work on

SURVEYING, HIGHWAY CONSTRUCTION, RAILROAD ENGINEERING, EARTHWORK, STEEL CONSTRUCTION, SPECIFICATIONS, CONTRACTS, BRIDGE ENGINEERING, MASONRY AND REINFORCED CONCRETE, MUNICIPAL ENGINEERING, HYDRAULIC ENGINEERING, RIVER AND HARBOR IMPROVEMENT, IRRIGATION ENGINEERING, COST ANALYSIS, ETC.

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HE editors have freely consulted the standard technical literature of America and Europe in the preparation of these volumes. They desire to express their indebtedness, particularly, to the following eminent authorities, whose well-known treatises should be in the library of everyone interested in Civil Engineering.

Grateful acknowledgment is here made also for the invaluable cooperation of the foremost Civil, Structural, Railroad, Hydraulic, and Sanitary Engineers and Manufacturers in making these volumes thoroughly representative of the very best and latest practice in every branch of the broad field of Civil Engineering.

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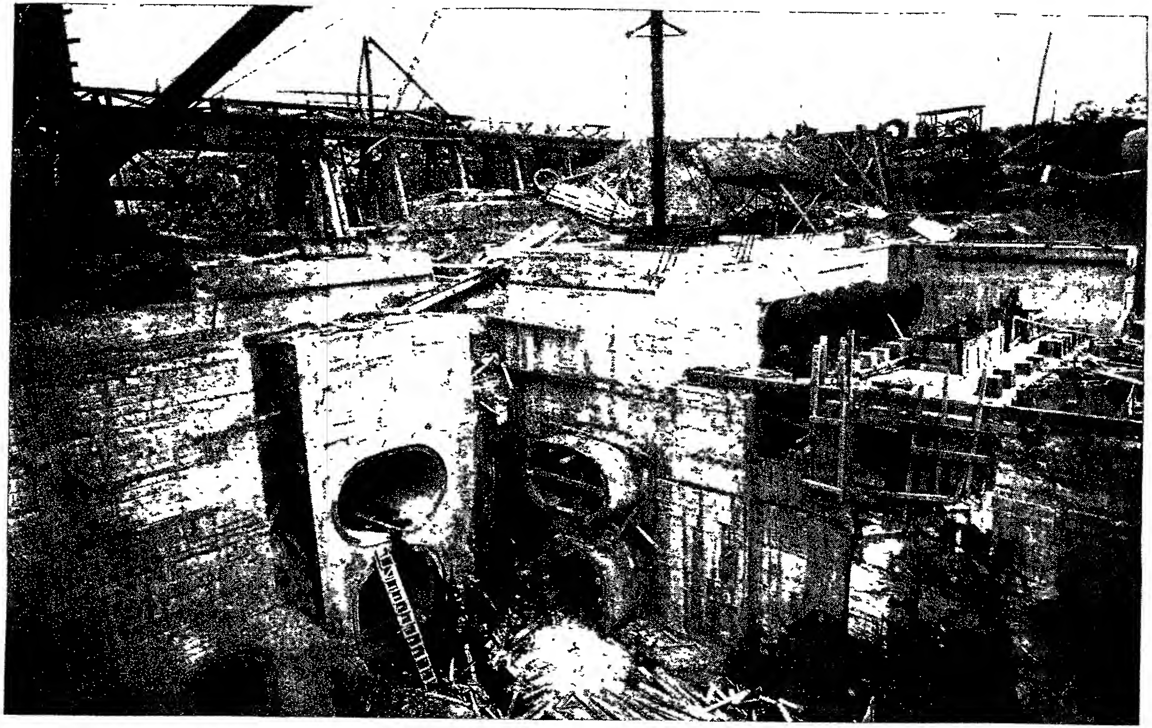


REINFORCED CONCRETE WORK ON LOWER GATE CHAMBERS FOR ASHOKAN DAM PROJECT
Photo by Brown Brothers, New York City

OF all the works of man in the various branches of engineering, none are so wonderful, so majestic, so awe-inspiring as the works of the Civil Engineer. It is the Civil Engineer who throws a great bridge across the yawning chasm which seemingly forms an impassable obstacle to further progress. He designs and builds the skeletons of steel to dizzy heights, for the architect to cover and adorn. He burrows through a great mountain and reaches the other side within a fraction of an inch of the spot located by the original survey. He scales mountain peaks, or traverses dry river beds, surveying and plotting hitherto unknown, or at least unsurveyed, regions. He builds our Panama Canals, our Arrow Rock and Roosevelt Dams, our water-works, filtration plants, and practically all of our great public works.

¶ The importance of all of these immense engineering projects and the need for a clear, non-technical presentation of the theoretical and practical developments of the broad field of Civil Engineering has led the publishers to compile this great reference work. It has been their aim to fulfill the demands of the trained engineer for authoritative material which will solve the problems in his own and allied lines in Civil Engineering, as well as to satisfy the desires of the self-taught practical man who attempts to keep up with modern engineering developments.

¶ Books on the several divisions of Civil Engineering are many and valuable, but their information is too voluminous to be of the greatest value for ready reference. The Cyclopedia of Civil Engineering offers more condensed and less technical treatments of these same subjects from which all unnecessary duplication has been eliminated; when compiled into nine handy volumes, with comprehensive indexes to facilitate the looking up of various topics, they represent a library admirably adapted to the requirements of either the technical or the practical reader.

¶ The Cyclopedia of Civil Engineering has for years occupied an enviable place in the field of technical literature as a standard reference work and the publishers have spared no expense to make this latest edition even more comprehensive and instructive.

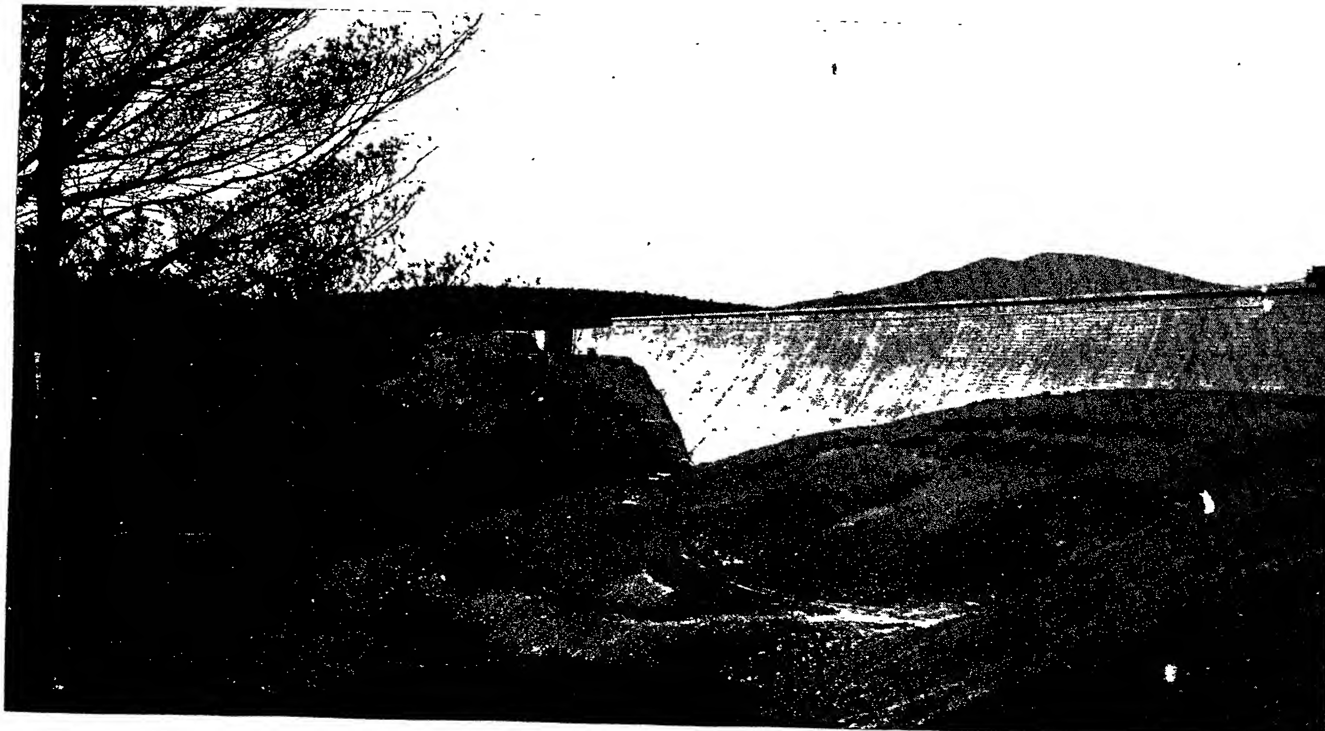
¶ In conclusion, grateful acknowledgment is due to the staff of authors and collaborators—engineers of wide practical experience, and teachers of well recognized ability — without whose hearty co-operation this work would have been impossible.

Table of Contents

VOLUME VII

*For page numbers, see foot of pages.

†For professional standing of authors, see list of Authors and Collaborators at front of volume.



ASHOKAN DAM, CATSKILL AQUEDUCT, PROVIDING PART OF NEW YORK CITY'S WATER SUPPLY
Photo by Brown Brothers, New York City

WATER SUPPLY.

PART I.

INTRODUCTION.

1. Historical. The earliest method of artificially obtaining a water supply was by the digging of wells. These were at first mere shallow cavities scooped out of the ground in low places; but it is interesting to know that the sinking of deep wells through rock dates from a very early period, the Chinese having been familiar with such work from very early times. Besides wells, other works for water-supply purposes were constructed by the Ancients, such as reservoirs, cisterns, aqueducts, etc.

The greatest development of waterworks construction in ancient times took place during the prosperous period of the Roman Empire, some of the finest works having been built at this time. To supply the chief cities of the empire great aqueducts were constructed, many miles in length, and there were in some cases several such aqueducts supplying a single city. Rome was at one time supplied from fourteen different aqueducts some of which had a length of 40 miles. The first of these was built about 312 B.C. and the last about 305 A.D. Some of the other cities which were well supplied with water at this time were Paris and Lyons in France, Metz in Germany, and Segovia and Seville in Spain.

The distribution of water in this age was by no means general. From the aqueducts the water first passed into large cisterns, and from these it was distributed through lead pipes to the fountains, baths and various public buildings, and to a few private consumers. The masses of the people were obliged to get their supply from public fountains. While the actual amount of water used by private consumers was not great the liberality of the supply for public purposes was so great that the total consumption was in many cases very high, some estimates making the consumption of water in Rome as high as 300 gallons per capita daily.

After the fall of Rome the entire subject of water supply was neglected for many centuries, and as one result, Europe was ravaged

many times by terrible pestilences, due to polluted water. In some cases even the purpose for which the ancient aqueducts had been built was forgotten by the inhabitants.

The development of modern waterworks began in Paris and London as early as the beginning of the 17th century, but little progress was made until the application of steam to pumping engines, first made in London in 1761. Since 1800 the development has been very rapid, both in Europe and America.

The first works in America for the supply of water to towns were those of Boston, built in 1652. Machinery was first used for pumping water at Bethlehem, Pennsylvania, where the works were put into operation in 1754. The first use of the steam engine was at Philadelphia in 1800, and in New York steam was applied in 1804. The principal development in this country has taken place since 1850, about ninety-eight per cent of all existing works having been constructed since that time. Nearly all towns of 2,000 inhabitants or more now have a public water supply, and the construction of works is progressing rapidly in many smaller towns and villages. While there is more work yet to be done in this direction, the chief work of the future will be in providing increased supplies for the rapidly growing cities and towns of this country, in developing new and better sources of supply and in the improvement of the quality of the existing supplies. There is also much opportunity for the engineer in the management of waterworks, in the direction of reducing cost of operation, prevention of waste and in the improvement of service in many other ways.

2. Value and Importance of a Public Water Supply. The most important use of a public water supply is that of furnishing a suitable water for domestic purposes. For such use the prime requisite is that the water should be pure. The transmission of certain diseases such as cholera and typhoid fever by polluted water is now universally recognized, and the value to a city of a pure supply when compared to one constantly polluted by sewage can scarcely be overestimated.

Another highly important function of a water supply is that of furnishing the necessary flushing water for a sanitary system of drainage. The most satisfactory and economical method yet found

for disposing of the organic wastes of a community is by the water-carriage system. Such a sewerage system is manifestly of but slight value to the public at large without the coexistence of a public water supply, as otherwise the necessary water for the flushing of closets—the most important function of a sewerage system—can be afforded by but few.

Besides furnishing an improved supply from the sanitary standpoint, a public works may often be made to furnish a water which for other reasons will be of greatly increased value to the domestic consumer; such as a soft water in place of a hard well water,—a point of very considerable importance to both domestic and commercial users.

A good water supply is also of great value to the manufacturing interests of a town. Many establishments, such as sugar refineries, starch factories, cleaning and dyeing houses, chemical works, etc., require an abundant water supply, and in some cases water of a high degree of purity. The question of water supply indeed often determines the location of factories. Large quantities are also used for operating elevators, for boiler purposes, and for many other uses that may be classed as commercial.

The most important public use of water supply is in extinguishing fires. The economic value of a good fire-protection system is directly shown in the reduced rates of insurance which follow its introduction or improvement. Instead of distributing a heavy fire loss among the people of a community through high rates of insurance it is assuredly much better economy to contribute to the maintenance of a public waterworks, which at the same time provides a suitable water for other purposes. To permit of the establishment of a certain class of factories it is absolutely essential that an efficient fire protection be furnished.

Other important public uses of a water supply are in street sprinkling and sewer flushing, in furnishing water for public buildings, and for drinking and ornamental fountains. A real value exists in the improved appearance which may be given a city by the use of water in fountains and for lawns and public parks; and, indeed, all the benefits accruing from a good water supply act indirectly to increase the desirability of a town for many purposes and to enhance the value of the property therein.

CONSUMPTION OF WATER.

3. General Considerations. When a new or enlarged water supply is under consideration one of the first questions to be answered is that relating to the quantity of water which will be required in the near future. The knowledge which is required includes not only the average daily quantity which will be needed, but also the monthly, daily, and hourly variation in the rate of consumption. In designing certain parts of the works the average consumption for the year is sufficient, but in certain other parts, such as pumps and distributing pipes, we need to know the greatest rate of consumption for a very short period of time.

There are many influences which affect the rate of consumption per capita of any given town or city. One of these is the actual population of the town. Thus in large cities the use of the public supply is almost a necessity, while in small towns and villages the private supplies may remain in use to a large extent long after the introduction of the public water supply.

The nature of the industries of a town is a large factor in determining the amount of water used; also the wealth and habits of the people, and the extent to which water is used for fountains, watering of lawns, street sprinkling, and other public purposes. Climate has also a very considerable influence, especially as to the amount used for sprinkling purposes and that which is wasted in winter to prevent freezing. It is probable, however, that the most important factors in determining the consumption is the degree of care taken to detect leakage or waste, and the fact as to whether the water is sold by measure or otherwise. Good quality, abundant quantity, and high pressure tend to increase the consumption by encouraging a more liberal use and often, at the same time, greater wastefulness.

4. The Average Daily Consumption Per Capita. In Table No. 1 are given the rates of consumption per capita for several American cities and towns in 1895.

It will be noted from Table No. 1 that a great variation exists in the rate of consumption in different cities and that the consumption in some of the cities is very high. For example, it is 271 gallons in Buffalo, New York, and 247 gallons in Allegheny, Pennsylvania. It will also be noted from a comparison of Tables No. 1 and 2 that the consumption is, on the average, much less in European than in

TABLE I.
Consumption of Water in American Cities and Towns.

City.	Population. 1900.	Daily consumption per inhabitant. 1895.
New York	3,437,202	100
Chicago	1,698,575	139
Philadelphia	1,293,697	162
Brooklyn	....	89
St. Louis	575,238	98
Boston	560,892	100
Cincinnati	325,902	135
San Francisco	342,782	63
Cleveland	381,768	142
Buffalo	352,387	271
New Orleans	287,104	35
Washington	278,718	200
Montreal	....	83
Detroit	285,704	152
Milwaukee	285,315	101
Toronto	....	100
Minneapolis	202,718	88
Louisville	204,731	97
Rochester	162,608	71
St. Paul	163,065	60
Providence	175,597	57
Indianapolis	169,164	74
Allegheny	129,896	247
Columbus	125,560	127
Worcester	118,421	66
Toledo	131,822	70
Lowell	94,969	82
Nashville	80,865	139
Fall River	104,863	35
Atlanta	89,872	42
Memphis	102,195	100

In Table No. 2 are given the rates of consumption for several European cities.

TABLE 2.
Consumption of Water in European Cities.

City.	Estimated population.	Daily consumption per capita, gallons.
London	5,700,000	42
Manchester	849,093	40
Liverpool	790,000	34
Birmingham	680,140	28
Bradford	436,260	31
Leeds	420,000	43
Sheffield	415,000	21
Berlin	1,427,200	18
Breslau	330,000	20
Cologne	281,700	34
Dresden	276,500	21
Paris	2,500,000	53
Marseilles	406,919	202
Lyons	401,930	31
Naples	481,500	53
Rome	437,419	264
Florence	192,000	21
Venice	130,000	11
Zurich	80,000	60

American cities. Both of these variations are due largely to the variation in practice in the use of meters to measure the water used and to charge accordingly. In some American cities meters are quite generally used, and without exception the consumption of water in those places is comparatively low. Meters are also generally used in European cities with the results as indicated in the table. It is true, however, that there is a greater general use of water for proper purposes in this country than in foreign countries.

5. Consumption of Water for Different Purposes. In studying the subject of the consumption of water it is desirable to consider the different uses of water under the following heads: (1) Domestic use; (2) Commercial use; (3) Public use; (4) Loss and waste.

(1) *Domestic Use.* Statistics collected from many sources where the supply has been actually measured by meter show that the amount of water used for domestic purposes will vary from about 15 to 40 gallons per capita; usually from 20 to 30 gallons. Where the supply is not metered, but is paid for according to the number and kind of fixtures in use, or the number of rooms in the house, the consumption may be several times the above figures. It has been known in some cases to go as high as 175 and 200 gallons per capita. Under these conditions it is difficult to predict what the consumption will be.

(2) *Commercial Use.* Under this head are included all uses for mechanical, trade, and manufacturing purposes. Large users of water for such purposes are office buildings and stores, hotels, factories, elevators, railroads, breweries, sugar refineries, and a few other industries. In large cities the use for commercial purposes is likely to be more than in small cities. Various statistics show a consumption for these purposes of 10 to 40 gallons per capita. The nature of the industries will determine very largely this item.

(3) *Public Use.* This includes the water used for schools and other public buildings, street sprinkling, water troughs and fountains, sewer flushing and the flushing of water mains, fire extinguishment, and a few other occasional uses. Water for such purposes is seldom measured, but the amount is not likely to exceed on the average a few gallons per capita, although the rate of consumption is far from being uniform. The water used for street-sprinkling purposes is likely to be quite a large proportion of the total, as much as 10 gal-

lons per capita being used in some places. The average is, however, not more than one or two gallons per capita. For fire purposes the total consumption is relatively small, but during fires the rate of consumption is very high for a short time. The total consumption for public purposes may be estimated from 3 to 10 gallons per capita.

(4) *Loss of Water.* The chief cause of waste is bad plumbing and carelessness on the part of the private consumer, but this source of waste has already been mentioned under the first item. There is in addition considerable waste due to leakage of mains and reservoirs and minor uses of water, not included under the foregoing. It is estimated that at least 15 gallons per capita should be allowed for this item.

From the foregoing analysis it may be concluded that a reasonable estimate of the consumption of water where meters are largely used will be about 40 gallons as a minimum and 120 gallons as a maximum; 75 or 80 gallons may be taken as a fair allowance under average conditions. Where meters are not used extensively the statistics in Table No. 1 show that 200 gallons per capita would not be an excessive figure, but it is impossible under such circumstances to make a very close estimate.

6. Variations in Consumption. The foregoing sections have discussed only the average consumption throughout the year. There will now be considered the variations which occur in the consumption from time to time.

Monthly Variations. In nearly all cases the rate of consumption reaches a maximum in the summer owing to the use of water for street and lawn sprinkling. This high rate usually extends over two or three months. A secondary maximum often occurs in the winter, due to the waste of water to prevent freezing, but the use of meters will largely prevent excessive variations from this cause. In extreme cases, however, the winter consumption may be very high. The monthly variations in consumption for several places are illustrated by the data given in Table No. 3.

From the table it may be concluded that the maximum monthly rate will seldom exceed 125 per cent of the average, it being in fact much below this figure for most places represented. Excessive consumption is likely to continue for two or three consecutive months, averaging for this longer period a rate of 110 to 115 per cent of the yearly average.

Daily Variations.—The maximum daily rate is usually estimated at about 150 per cent of the average. In Table No. 3 very considerable differences are to be noted in the ratios for different places, these being caused by a variety of conditions, some accidental and some constant.

TABLE 3.

Maximum Monthly and Daily Ratios Expressed as Percentages of Average Consumption.

City.	Ratio of maximum monthly to average consumption.	Ratio of maximum daily to average consumption.	City.	Ratio of maximum monthly to average consumption.	Ratio of maximum daily to average consumption.
Chicago	108	116	Louisville	127	135
Philadelphia	110	122	Columbus	107	157
Boston	114	119	Fall River	115	......
Cincinnati	124	153	Dayton	118	178
Cleveland	114	146	Newton	125	143
Buffalo	......	168	Pawtucket	111	153
Detroit	117	150	Woonsocket, R. I.	122	155
Milwaukee	113	......	Marquette, Mich	139	194

The maximum daily rate will usually occur in the month of maximum consumption, and a rate considerably above the average for the month will occur for several consecutive days. Thus where the maximum daily consumption is 150 per cent of the average, the maximum weekly consumption is likely to be from 130 per cent to 140 per cent of the average, but for longer periods of time the rate will approach the monthly maximum.

Ordinary Hourly Variations. If there were no waste or leakage, the consumption during several hours of the night would be almost nothing and the consumption during several hours of the day would be two or three times the average for the twenty-four hours. It is a fact, however, that the rate of consumption at night is usually as much as 60 per cent of the average, and during the hours of maximum consumption it is not often more than one and a half times the average. Where waste is carefully prevented, and the consumption therefore low, the variation during the twenty-four hours will be relatively greater than where the waste is great and the total con-

sumption great. Fig. 1 shows typical curves representing the hourly variation in consumption throughout the day. The curve for New York illustrates what occurs in a city where waste is fairly large, while that for Des Moines represents a case where consumption is small and the waste largely prevented. The average daily consumption per capita for New York was 100 gallons and for Des Moines 43 gallons.

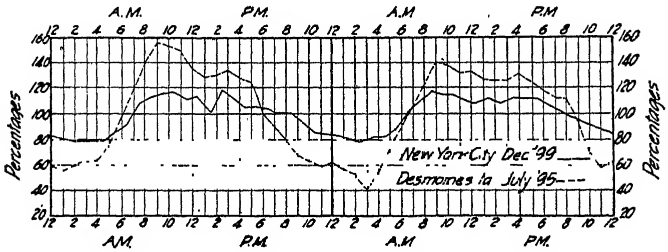


Fig. 1. Typical Curves Showing Hourly Variations of Water Consumption.

Consumption for Large Fires. The consumption for large fires must be considered in addition to the rates given above, The maximum rate of fire consumption in gallons per capita per day for a town or city of average character may be taken equal to $\frac{1000}{\sqrt{x}}$, where $x =$ population in thousands. This is based on Mr. Kuichling's estimate of the required number of fire streams.

If, for example, the average consumption is 100 gallons per capita, then the fire rate in per cent of the average will be as follows for different size cities:

Population.	Rate of fire consumption in percentage of average, when average equals 100 gallons per day.
1,000	1000 per cent.
5,000	447 "
10,000	316 "
50,000	141 "
100,000	100 "
200,000	71 "
300,000	58 "
500,000	45 "

For other values of the daily consumption the percentages would vary accordingly, being greater for smaller consumptions. In the case of small cities the fire rate is evidently the principal factor to be considered; in large cities it is of much less relative importance. The

duration of the above rate of fire consumption may be several hours; it has been estimated by Freeman at about six hours as a maximum.

The Combined Maximum Hourly Rate. In obtaining the total maximum rate of consumption at times of fires it is not necessary to assume that a great fire will occur coincident with the maximum use for other purposes. In fact, at times of great fires the use of water for many purposes would be interrupted. For average conditions the following may be taken as a reasonable allowance:

If the average daily rate is 100 per cent, then the maximum daily rate equals 150 per cent, and adding 20 per cent for increased consumption during day gives a total of 180 per cent. To this the fire consumption should be added by the use of the formula of the preceding paragraph.

Example. If the average daily consumption of a city of 20,000 inhabitants equals 80 gallons per capita, what will be the approximate maximum rate of consumption (a) for the day of greatest consumption, and (b), at the occurrence of a large fire?

From the foregoing discussion the maximum daily consumption may be estimated at 80×150 per cent $= 120$ gallons per capita.

From the above estimate the rate for ordinary use may be taken at 180 per cent of 80 gallons or $80 \times 180 = 144$ gallons. The fire rate $= \frac{1000}{\sqrt{20}} = 224$ gallons per capita. The total rate is therefore $144 + 224 = 368$ gallons per capita per 24 hours.

7. Growth of Cities. A necessary factor in any estimate of future consumption is that of future population. The rate of growth of different cities is exceedingly various, but of any one city it is likely to be fairly constant for several years, or at least will vary but slowly. The older and the larger the city the more uniform the rate of growth, and, barring national disasters, a fairly close estimate can be made for two or three decades in the future.

Probably the best way to estimate the population of a city for several years in the future is to take as a basis its growth in past years. If conditions do not change the per cent added each year or decade is apt to remain about the same. No one can predict closely the growth of a city, and in water-supply problems a close estimate is unnecessary.

Example. If the population of a city be 5,250 in 1880, 7,670 in 1890 and 11,400 in 1900, estimate the population in 1910.

The growth from 1880 to 1890 was 2,420, equal to 46 per cent, and from 1890 to 1900 it was 3,730, equal to about 49 per cent of the population in 1890. These figures show a steady growth, and it may be assumed that for the next decade the growth will be about the same, say 48 per cent. Then, 48 per cent of 11,400 = 5,472, and the estimated population in 1910 = 11,400 + 5,472 = 16,872.

The population for 1920 may be estimated in the same way, but the result will be much more uncertain than for 1910.

SOURCES OF SUPPLY.

8. Classification. The sources of water supply may be divided into the following classes, according to the general source and the method of collection:

A. Surface waters:
1. Rain water collected from roofs, etc.
2. Water from rivers.
3. Water from natural lakes.
4. Water collected in impounding reservoirs.

B. Ground waters:
5. Water from springs.
6. Water from shallow wells.
7. Water from deep and artesian wells.
8. Water from horizontal galleries.

Each of the above sources except the first and last is at present furnishing many cities in the United States with a more or less satisfactory water.

The following table gives the number of waterworks in 1896 obtaining their supply from the various sources indicated.

Region.	Surface waters.	Ground waters.	Total.
Northeastern States	615	511	1,238
Southeastern States	143	180	340
North Central States	193	469	715
Western States	329	662	1,063
Total	1,280	1,822	3,356

SURFACE WATER SUPPLIES.

9. Rainfall is the source of all water supply, whether it be caught as it flows over the surface or is first allowed to percolate into the ground to furnish water for wells and springs. The amount

of rainfall is expressed in inches of depth upon a horizontal surface, snowfall being reduced to its equivalent amount of rainfall. With the ordinary rain gauge it is impracticable to determine rates of rainfall for short periods of time, the records usually obtained from these gauges being merely the total amounts of rainfall for each twenty-four hours. For estimating flood volumes from small areas, however, it is important to know the rate of rainfall for much shorter periods than one day. For this purpose self-recording gauges are essential, that is, gauges which give a continuous record of the rainfall or a record taken at such short intervals as to be for all practical purposes continuous. Various forms have been devised, some weighing the water, others recording by volume.

Rainfall statistics for a large number of stations can now be readily obtained from the monthly reports of the Weather Bureau. The data of importance in connection with water-supply questions are the mean yearly rainfall, the deviation from this in dry years, the monthly rainfall, and finally the maximum depth of rain falling in a single day or less.

10. Mean Annual Rainfall. The mean annual rainfall for a number of stations in the United States is shown in Table No. 4. The table also gives the ratio of the rainfall in the driest year, covered by the statistics, to the average.

The maximum rainfall is along a narrow belt of the North Pacific coast, where it considerably exceeds 60 inches. Towards the interior the amount rapidly falls off, and between the Sierras and the Rocky Mountains it ranges from 5 to 15 inches. East of the Rockies there is a gradual increase eastward and southward to a maximum along the Gulf of 60 inches, and from 40 to 50 inches on the Atlantic coast. The table also shows that in the driest years the rainfall is in most places only 50 to 60 per cent of the average. In the central and Western States the variation is greater than in the Eastern States.

The monthly distribution of the rainfall is of great importance in all questions relating to the utilization of water for power purposes or for the supply of cities. The rain falling in the summer months, when vegetation is using a maximum of water and evaporation is rapid, is of but little value for supplying water to the streams. It is the winter and spring rains which must largely be relied upon to fill reservoirs and to raise the low ground water to its normal level.

TABLE 4.

General Rainfall Statistics for the United States.

Station.	Mean yearly rainfall, Inches.	Per cent rainfall, driest year to mean rainfall.
Boston	45.4	60
New York	44.7	62
Philadelphia	42.3	70
Charleston	49.1	48
Jacksonville	54.1	74
Shreveport	48.2	67
Mobile	62.6	68
New Orleans	60.3	64
Vicksburg	52.7	70
Louisville	47.2	74
Cairo	42.6	62
Cincinnati	42.1	60
Cleveland	36.6	71
Marquette	32.3	69
Chicago	34.0	66
Milwaukee	31.0	66
St. Louis	40.8	55
St. Paul	28.2	53
Duluth	30.7	65
Omaha	31.4	57
North Platte	18.1	56
Denver	14.3	59
Salt Lake City	18.8	55
Spokane	18.6	73
Santa Fe	14.6	53
Yuma	2.8	25
San Diego	9.7	30
Los Angeles	17.2	33
San Francisco	23.4	51
Portland	46.2	67

11. Maximum Rates of Rainfall. In estimating the maximum flood discharges of small streams—a matter of very great importance in the design of dams and reservoir embankments—it is desirable to know the maximum rates of rainfall for periods of a few hours or a single day. Great rainstorms occur but rarely, but in hydraulic works where a failure would mean not only the destruction of property but often a great loss of life, it is necessary to provide against the greatest flood ever likely to occur. Accurate data of such floods must be based on many years of observation, but extraordinary rainfalls are likely to occur almost anywhere, and it may be assumed that what has happened in one locality may happen at any place in the same region. Examination of the data contained in the United States Weather Bureau Reports shows that in the Northern and Central States a rainfall at the rate of 4 inches for

one hour and 8 inches for 24 hours represents the greatest rain likely to occur; in the South Atlantic and Gulf States these figures should be about 4 inches for one hour and 10 inches for 24 hours.

That excessive rainfalls are of sufficient extent to cover areas of such size as are ordinarily considered in water-supply problems is shown by the statistics of great storms. In October, 1869, a great storm occurred in the eastern part of the United States, with its maximum intensity in Connecticut. A careful analysis of the records made by Mr. James B. Francis shows the areas covered by different depths of rain to have been as follows:

Depth of rain.	Area covered.
6 inches or more	24,431 square miles.
7 " " "	9,602 " "
8 " " "	1,824 " "
9 " " "	1,046 " "
10 " " "	519 " "
11 " " "	179 " "

The following are some of the maximum rates observed in this storm:

4.00	inches in	2	hours.
4.27	"	3	"
5.86	"	18.5	"
7.15	"	24	"
8.90	"	30	"
8.44	"	42	"

FLOW OF STREAMS.

12. When a stream is under consideration as a source of water supply, the peculiarities of its flow—the minimum, maximum, and total flow for various periods of time—are among the first things to be determined. The most accurate as well as the most direct method of determining these is by means of a series of gaugings extending over several years, but, where gaugings are not to be had, or where they are very limited in extent, as close an estimate as possible must be made from a comparison with other streams whose flows are known, taking into account as far as may be the differences in rainfall, climate, and in the various characteristics of the different watersheds.

Rainfall is expressed in inches in depth, and the rate in inches per hour or per twenty-four hours; and for comparative purposes stream flow is often likewise expressed, meaning thereby inches in depth over the entire watershed. For other purposes the flow is usually expressed in cubic feet, or cubic feet per square mile of water-

shed, and the rate of flow in cubic feet per second, or cubic feet per second per square mile. The foot and second units are also convenient to use in all hydraulic formulas, but in matters pertaining to storage and distribution the gallon unit is in common use, and rates are expressed in gallons per minute and gallons per twenty-four hours.

For convenience in computations relative to rainfall and flow of streams, the following table is inserted:

TABLE 5.

Volumes and Rates of Flow in Feet and Seconds Corresponding to Given Volumes and Rates of Rainfall in Inches and Hours.

Depth in inches.	Cubic feet per square mile.	Inches per hour.	Cubic feet per second per square mile.	Inches per 24 hours.	Cubic feet per second per square mile.
0.1	232,320	0 1	64.5	1	26.9
0.2	464,640	0.2	129.0	2	53.8
0.3	696,960	0.3	193.5	3	80.7
0.4	929,280	0.4	258.1	4	107.5
0.5	1,161,600	0.5	322.6	5	134.4
0.6	1,393,920	0.6	387.1	6	161.3
0.7	1,626,240	0.7	451.7	7	188.2
0.8	1,858,560	0.8	516.2	8	215.1
0.9	2,090,880	0.9	580.7	9	242.0
1.0	2,323,200	1.0	645.3	10	268.9

One inch of rain = 2,323,200 cu. ft. per sq. mile.
One inch per hour = 645.33 cu. ft. per sec. per sq. mile.
One inch per 24 hours = 26.89 cu. ft. per sec. per sq. mile.
One cubic foot = 7.4805 U. S. gallons.
One cubic foot per sec. = 646,300 gallons per day.

The question of the flow of streams naturally divides itself into three parts:

First, the minimum flow of the stream.

Second, the maximum or flood flow.

Third, variations in the flow through successive months and years.

The first information is necessary in case a stream is under consideration for which but little storage is obtainable, or in answer to the question whether it is practicable to draw directly from the stream without storage. The second is of great importance in the design and execution of all river work, and especially in determining the size of waste weirs. The third determines the supplying capacity of the watershed and the size of impounding reservoirs.

EXAMPLES FOR PRACTICE.

1. If a rain is falling at the rate of $\frac{1}{2}$ inch per hour, how many cu. ft. per sec. will this amount to over an area of 10 sq. mi.? 3,226 cu. ft. per sec. Ans.

2. If 1 inch of water is collected from an area of 20 sq. mi., how many days will this supply a town of 15,000 inhabitants using 100 gallons per capita daily?

The total amount of water collected = 2,323,200 × 20 = 46,464,000 cu. ft. = 347,500,000 gal. This will last 231 days. Ans.

13. The Dry-Weather Flow. The dry-weather flow of streams is maintained entirely from ground and surface storage; and as facilities for such storage vary in different watersheds, so will the minimum flow vary.

In Table No. 6 are given the minimum flows of several streams in different localities. It will be seen that the minimum varies greatly with the size of the stream and locality, and that streams of several hundred square miles of drainage area may have a minimum of zero.

TABLE 6.

Minimum and Maximum Flow of Streams.

Stream.	Place.	Drainage area, square miles.	Minimum flow, cubic feet per sec. per sq. mile.	Maxim'm flow, cubic feet per sec. per sq. mile.
	New England.			
Merrimack	Lawrence	4,599	0.31	20.87
Connecticut	Hartford	10,234	0.51	20.27
Nashua	Massachusetts	109		104.5
Sudbury	Massachusetts	78	0.036	44.2
	New York.			
Chemung	Elmira	2,055		67.1
Croton West Br.		20.37	0.016	54.43
	New Jersey.			
Delaware	Stockton	6,790	0.17	37.5
Pequannock		48		115
	Pennsylvania.			
Perkiomen	Frederick	152	0.39	34.9
South Fork	Dam in Creole Township	48.6		215
	Maryland.			
Potomac	Cumberland	1,364	0.018	131
	Illinois.			
Rock	Rockford	6,500	0.0158	
Des Plaines	Riverside	630	0	21.4

14. Flood Flow. The maximum rate at which the waters from great storms will pass down a stream is affected largely by the steepness of the slopes, by the size and shape of the drainage area, and by the distribution of the branches. Small areas will have larger maximum rates of flow than large areas, other things being equal, as the former are affected by short rainfalls of high rates, while in the latter case the maximum flows are caused by rains of longer duration but of less intensity. For a like reason streams with steep slopes will have a higher maximum rate than those with flat slopes.

Of great importance in distributing the run-off over a long interval of time, and so reducing the maximum rate, is the surface storage of natural lakes and ponds and of those created by the inundation of large flats bordering the stream. The effect of this last factor may be sufficient to reduce the flood flow to one-half or one-fourth that of a stream with a narrow valley.

In Table No. 6 great variation in the maximum flow is observable, due partly to the varying rates of rainfall, but largely to the different characteristics of the streams. Various formulas have been proposed for expressing the maximum flow of a stream, some involving only the rainfall and area, while others attempt to take account also of the slope and shape of the watershed.

Among the most widely known of this class of formulas is that given by Fanning and recommended by him as applicable to average New England and Middle-State basins. It is

$$Q = \frac{200}{\sqrt[6]{M}} \qquad \textbf{(1)}$$

in which Q = discharge in cubic feet per second per square mile and M = area in square miles. It gives results probably somewhat too low for small areas.

Example. What will be the flood flow according to formula 1 for a drainage area of 10 square miles?

The flow will equal $\frac{200}{\sqrt[6]{10}}$ = 136 cu. ft. per sec. per sq. mi.

TABLE 7.

Statistics of the Yearly Flow of Streams.

Stream.	Area drained, square miles.	Average yearly flow. Rain, inches.	Average yearly flow. Flow, per cent of rainfall.	Dry year flow. Rain, inches.	Dry year flow. Flow, per cent of rainfall.
Cochituate	18.87	47.08	43.2	31.20	31.3
Croton	338.0	48.38	50.8	38.52	37.8
Genesee	1,060	39.82	32.5	31.00	21.5
Perkiomen	152	47.98	49.2	38.67	40.4
Potomac	11,043	45.47	52.7	37.03	39.2
Savannah	7,294	45.41	48.9	43.10	37.7
Upper Mississippi	3,265	26.57	18.4	22.86	7.1

15. Annual Discharge. Table 7 gives some statistics of the annual flow of streams as compared to rainfall. It will be seen that in the dry years the percentage running off is much less than in the average year. From these data and other statistics it is estimated that for a stream of average conditions east of the Missouri and Mississippi Rivers the percentage of rainfall flowing off for different annual rainfalls is about as follows:

Rainfall, inches.	Per cent running off.
20	25 to 35
30	30 to 40
40	35 to 45
50	40 to 50

In the nature of the problem there is a wide variation in percentage due to variations in the conditions of the watershed, climate, etc. Whatever tends to promote evaporation from the watershed decreases the run-off. Thus a watershed with a large percentage in grass will yield a less amount than one with rocky and barren hillsides; one with a large percentage of water surface, less than one with a small percentage. Again, the higher the temperature the greater the evaporation and the less the stream flow. Steep, rocky hillsides will give a large per cent of the rainfall to the streams, but the flow will be very irregular; flat grass lands will give little or nothing to the streams during the season of growth. All these things must be considered in estimating the flow of a stream from rainfall data and from statistics of the flow of other streams.

Examples. Estimate the flow of a stream during dry years where the average annual rainfall is known to be 40 inches.

The rainfall for a very dry year may be taken from Table No. 4 at say 60 per cent of the average or $40 \times .60 = 24$ in. For a rain-

fall of this amount the per cent running off will probably be between 25 and 35. If this is an average watershed we may put it at about 32 per cent. The run-off will then be estimated at $24 \times .32 = 7.7$ inches.

Note. The wide variation in percentage indicates that such estimates as this are very uncertain. Actual stream measurements are the only safe guide.

2. How much water can probably be collected in a dry year from an average watershed where the rainfall in very dry years is 30 inches?

By the estimates of section 15 it is probable that at least 33 per cent will run off or can be caught in a reservoir. This amounts to $30 \times .33 = 9.9$ inches.

By Table No. 5 this amounts in gallons per sq. mi. to $9.9 \times 2{,}323{,}200 \times 7.48 = 172{,}000{,}000$ gallons.

16. Monthly Variation in Stream Flow. During dry years very little water can be collected from summer rains. Dependence must be had on winter snows and spring rains for filling storage reservoirs and nearly all the yearly supply will be caught in the months from December to May inclusive. During average seasons a large proportion of the stream flow occurs in the summer months. Generally about three-fourths of the yearly flow occurs in the months from December to May and only one-fourth from June to November, whereas in very dry years the summer flow may be considered as practically nothing.

17. Quality of Surface Waters. Surface water supplies are drawn from two general sources—rivers and lakes. River supplies may be divided into those obtained directly from large rivers and those obtained from impounding the flow of small streams in reservoirs. The quality of surface waters may be considered with reference to: (1) appearance, (2) mineral content, (3) the presence of disease-producing organisms.

(1) The appearance of a water is affected by the presence of clay and sand in suspension, rendering the water turbid, and by certain vegetable material giving the water a distinct color. Turbidity varies according to the nature of a watershed. While a turbid water is very objectionable for household use it cannot be said to be actually dangerous. Turbidity is removed by allowing the water to rest in

reservoirs, thus permitting the clay to settle, or by passing the water through filters. Surface waters flowing through swampy regions are usually colored, due mainly to the extraction of soluble coloring matter from vegetable material. Such peaty waters, while perhaps unsightly in appearance, may be, however, perfectly wholesome in spite of this physical defect.

(2) While flowing surface waters do not dissolve so much mineral matter as ground waters, yet they take up an appreciable amount, depending considerably on the character of the soil over which they pass. A large part of the mineral content is usually carbonate of lime. In general surface waters are preferable to ground waters as regards their mineral content, a hard water (one containing lime) being less desirable for culinary and manufacturing purposes.

(3) The most important question relating to the quality of a water is whether it is dangerous to the health. It has been well demonstrated that certain diseases, particularly cholera and typhoid, are caused by certain minute organisms called bacteria. These inhabit the intestinal tract of persons sick with the disease and are present in enormous numbers in the sewage wherever these diseases exist. Whenever such sewage or drainage gets into the water supply of any town an outbreak of the same epidemic is sure to appear. Many cases are on record of whole villages being affected through the contamination of the water supply by a single diseased person. From such facts it is seen that the quality of a water supply from this point of view is exceedingly important.

A surface water supply can be absolutely safe only when it is drawn from an uninhabited area. A few scattered farm houses, if not located too near a water course, are not likely to cause serious pollution. But where the watershed is quite populous, and especially where villages are located in the valleys, the danger of the transmission of disease through the water supply is very great.

The danger in the use of water from a large stream depends on the amount and nearness of the pollution. All large streams receive more or less drainage from towns and cities, but if such pollution is relatively small and remote the danger is small. As a rule a surface water supply is not free from danger unless the water is artificially purified by some adequate means, but many large cities in the United States continue to use water supplies which are badly contaminated.

The result of such use is shown in the relatively high death rate from typhoid fever in such places.

The quality of lake supplies is likely to be better than that of rivers. Such water is usually quite free from turbidity, as the sediment brought into it by the tributary streams soon settles; and unless polluted by sewage in the immediate neighborhood, it is likely to be relatively safe from a sanitary point of view. Experiments show that in the settling of the clay and sand particles, the bacteria settle to a great extent, and a marked purification takes place in a polluted water. For the same reason that lake water is better than river water, it is true that a supply from a small stream is usually improved by storage in a large storage reservoir. Sometimes, however, vegetable growths occur in reservoirs which give to the water a disagreeable odor.

GROUND WATER SUPPLIES.

18. Occurrence of Ground Water. The rain which falls upon the ground is disposed of in three ways: A part flows off immediately in the streams, a part is evaporated from the ground and vegetation, and a part percolates into the soil.

Percolating water that escapes beyond the reach of vegetation must, in obedience to the law of gravitation, pass on downward until it reaches an impervious layer of some sort. The immediate impervious stratum is the surface of the water which has preceded it and which has in past ages filled every pore and crevice of the earth's crust up to a certain level at which the escape of the water laterally becomes equal to the addition from percolation. The accumulation of water which thus exists in the ground is called *ground water*, and its surface the *ground-water level* or the *water table.*

In limestone regions it is sometimes the case that quite large streams are found flowing underground, and large cavernous spaces may be converted into underground lakes of considerable size, as in the great caverns of Indiana and Kentucky. Such bodies of water are, however, rarely available for a water supply, and it may be taken as a safe rule for ground-water supplies dependence must be placed upon the water which percolates into and flows through the pore-spaces in soils and rocks, the amount of which is strictly dependent upon the rainfall and the laws of hydraulics that govern the flow.

19. General Form of the Water Table. Under the action of gravity the surface of the ground water always tends to become a level surface, and as long as a supply is maintained through percolation there will be a continual downward and lateral flow which will on the average be equal to the percolation. In surface streams a very light inclination is sufficient to cause a rapid movement of water, but in the ground the resistance to movement is so great that a steep gradient is necessary to maintain even a very low velocity.

If we imagine the ground to be throughout of uniform porosity, the ground-water surface will conform in general outline to the ground surface, but with less variations. Such an ideal condition is represented in Fig. 2. At the margin of streams as at A and B the level

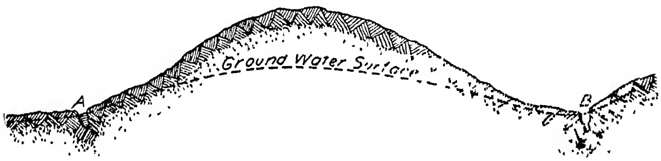


Fig. 2. Relation of Ground Water to Surface Water.

of ground and surface waters will coincide. Passing back from the stream the ground-water level will gradually rise, but at a less rate than the ground surface, then descend again into another depression, etc. In the valley there is also a fall parallel to the stream, corresponding to that of the surface water, and the direction of flow will be towards and slightly down the stream in the line of greatest slope.

Variations in ground-water level take place comparatively slowly, following gradually the variations in yearly, seasonal, and briefer periods of rainfall. Near streams and in lowlands the level varies little, being fixed largely by the level of the adjacent surface water. At higher points in the water table the level is subject to correspondingly great fluctuations, often many feet in extent. In porous material where slopes are small the variations are small.

20. Porosity of Soils. All soils and rocks near the surface of the earth are capable of absorbing more or less water. In sand of a fairly uniform size the porous space is commonly from 35 to 40 per cent of the entire volume. Mixed sand and gravel will have a smaller percentage of voids, the decrease depending on the variation in size of particles, but it will seldom be less than 25 per cent. Rocks

will vary in porosity from a very small fraction of 1 per cent in the case of some granites to 25 or even 30 per cent for some loose textured sandstones.

The amount of moisture which a soil or rock will absorb is, however, not of so much importance to the water-works engineer as is the carrying capacity and the amount which can readily be drawn from such material when previously saturated. In fine soils the movement of the water is so slow and such a large part of the water is retained by capillary action that such soils are of little value as carriers of water; and to obtain economically the large quantities required for public supplies it is necessary that the water-bearing material be of a very open, porous character. Adequate supplies are rarely obtained from anything but sand and gravel deposits, or from very porous rock. The most favorable formations for furnishing large quantities of water, are the various sandstones, conglomerates, and gravel deposits. Sandstones are found which vary in texture from a very compact rock having a very small degree of porosity to a material almost as porous as sand. Uncemented sands and gravels are of course the most favorable as regards porosity, but they are apt to be rather limited in extent.

21. The Flow of Ground Water. It has been explained in the previous section that the water in the ground has in general a slow rate of flow through the ground. Where a supply of ground water of considerable amount is to be obtained this rate of flow is of much importance. To get water from a ground-water "stream" is exactly similar to the taking of water from a surface stream; in both cases the flow of water in the stream must be at least equal to the proposed draught or the supply will be inadequate. The notion is quite common that in many places the water in the ground is inexhaustible. This is an entirely mistaken idea as is well illustrated by the gradual failure of many ground-water supplies.

Ground water in large quantities is usually obtained either from large gravel deposits of comparatively small depth, forming broad underground streams, or from extensive deposits of porous rock like sandstone, the latter source being tapped by deep wells many of which are the well known "artesian" wells. In the case of a gravel deposit near the surface it is often possible to estimate the quantity of water actually flowing through a given section of the deposit.

The best method of estimating capacity of a ground-water source is by means of actual pumping tests carried on for a sufficient length of time to bring about an approximate state of equilibrium between the supply and the demand which will be shown when the level of the water in the trial well ceases to lower. It will rarely be practicable to continue such tests until perfect equilibrium is reached, for in many cases several years of operation would be required to determine the ultimate capacity of a source. Pumping tests of short duration are apt to be very deceptive, as the ground water may exist in the form of a large basin or reservoir with very little movement, corresponding to a surface pond with small watershed, and brief tests would give but little more information than similar tests on a pond.

Where it can be done it is very desirable to get an approximate idea of the amount of water actually flowing per unit of time through the area in question.

To do this we must estimate the velocity of flow, the cross-section of the porous stratum containing the water, and the percentage of porous space.

The rate of flow of ground water streams is very small compared to that of surface streams. It depends on the slope or inclination of the ground and upon the size of the grains of sand or gravel through which it passes. The following table shows about what the velocities are likely to be for various slopes and conditions of soil.

TABLE 8.

Velocities of Flow of Ground Water in Feet Per Day.

Material.	Slope of ground, feet per mile.					
	10	20	30	40	50	100
Fine Sand	0.2	0.4	0.6	0.8	1.0	2.0
Medium Sand	1.5	3.0	4.5	6.0	7.5	15.
Coarse Sand	4.0	8.0	12.0	16.0	20.	40.
Fine Gravel, free from sand ...	20–40	40–80	60–120	80–160	100–200	200–400

The velocity of flow having been determined it remains to estimate the actual quantity of water flowing through a given territory. Of the total volume of a body of sand or gravel, the water will occupy only about 25 to 30 per cent. The actual volume of

water, therefore, which will pass through a given section will be only 25 or 30 per cent of the amount were it solid water. If v = velocity of flow in feet per day, A = area of cross-section of the porous bed at right angles to the direction of flow, then assuming a porosity of 25 per cent or $\frac{1}{4}$th, the actual volume of flow per day will be in cubic feet

$$Q = \frac{1}{4} vA \qquad (2)$$

Thus suppose we have a porous bed of coarse sand in which the water is 10 feet deep, the bed is 500 feet wide and slopes 20 feet per mile. The velocity of flow by Table 8 may be taken at about 8 feet per day. The cross-section $A = 10 \times 500 = 5{,}000$ square feet. Hence the volume of flow will be approximately $\frac{1}{4} \times 8 \times 5{,}000 = 10{,}000$ cubic feet per day, or about 75,000 gallons. This being the total rate of flow through the sand it is evidently the greatest amount of water that could be extracted from this sand deposit by means of any system of wells or other devices. To give the above results the bed must be of considerable length and the water in it must be about the same depth throughout and have the same slope as the surface.

SPRINGS.

22. Formation of Springs. Springs are formed where, for any reason, the ground water is caused to overflow upon the surface. The conditions causing their formation are varied and should be carefully studied in connection with the design of collecting-works, as upon them depend largely such questions as the constancy of flow, the possibility of increasing the yield by suitable works, and the probable success of a search for additional springs. According to differences in these conditions springs may be divided into three general classes, each of which will be discussed separately.

First Class. The most important class of springs is that in which the water, in its lateral movement, is brought to the surface at the outcrop of a porous stratum where it is underlain by a relatively impervious one. Fig. 3 represents such conditions, the ground water escaping at the outcrop of the impervious material thus forming a spring. The porous stratum may be sand or gravel, or a porous rock; while the impervious layer is usually clay, or rock of an argillaceous character.

There are many cases of large springs of this class, the supplies for some of the largest cities of Europe being obtained from such sources. The city of Vienna is supplied from springs 60 miles distant that occur at the outcrop of a fractured dolomitic limestone underlaid by slate. The largest spring, the Kaiserbrunnen, has an average flow of about 150 gallons per second, varying from 60 to about 250.

Second Class. Under this class are considered those springs where the water-bearing stratum is covered to a greater or less extent by an impervious one, and which are therefore more or less artesian in character. In this case the water finds its way to the surface

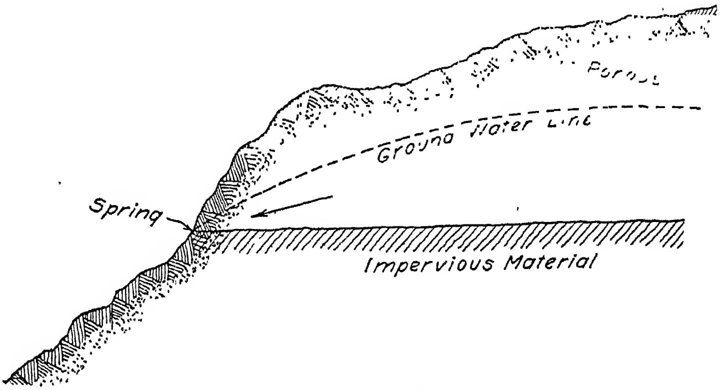


Fig. 3. Formation of Springs.

where the overlying impervious material is broken, or through a fault, or it breaks through at places where it is not sufficiently strong or compact to resist the upward pressure.

In some cases springs of this character are fed by water coming long distances through extensive formations which at other points offer conditions favorable for artesian wells. Conditions of this sort give rise to the peculiar phenomenon of large fresh water springs which boil up in the ocean several miles out from the Florida coast, and it is supposed that the great springs in northern Florida are from a similar cause.

Third Class. The third class of springs are mere overflows of the ground water, and occur whenever the carrying capacity of the porous material is insufficient to convey the entire tributary flow. Such conditions also give rise to marshy places at the foot of hills and even on side hills.

23. The Yield of Springs. The yield of any particular spring can readily be determined by weir measurements, and if these are carried out through a period of drought they will give all needed information regarding the supplying capacity of the existing spring.

Springs of the first class will vary in yield with the variations in ground-water level and, therefore, will vary with the rainfall, but will not wholly cease to flow if the water is intercepted by suitable constructions.

Springs of the second class are apt to be much less affected by variations in rainfall than either the first or the third class.

Where a spring of this class exists, investigation may show that the ground-water stream from which it is fed is of considerable size and that the water of the spring is but a small portion of the entire flow. In such a case the yield may be increased by simply enlarging the opening, or by sinking wells and pumping therefrom, as in the case of an ordinary ground-water supply.

Springs of the third class are liable to very great fluctuations, the flow often ceasing entirely.

ARTESIAN WATER.

24. General Conditions. Whenever a water-bearing stratum dips below a relatively impervious one the former becomes in a sense a closed conduit or pipe, and if the flow out of this conduit at the lower end be impeded from any cause, the water will accumulate and exert more or less pressure against the impervious cover The amount of this pressure will depend on the extent to which the flow is obstructed and on the elevation of the upper end of the conduit, that is, of the outcrop of the porous stratum. If a well be sunk through this impervious stratum at any point, the water will rise in it in accordance with the pressure; and if the surface topography and pressure are favorable, the water may rise to the surface, or considerably above, in which case the well becomes a true artesian, or flowing, well.

Fig. 4 shows an ideal condition for artesian or flowing wells. If A B is a porous stratum outcropping at A and B and covered by an impervious stratum of clay or impervious rock, water entering at A could escape at the lower end B, but at intermediate points would exert a pressure on the covering. If the resistance to flow

were uniform, and no water could escape except at B, the decrease of head from A to B would be uniform, or in other words the hydraulic grade line would be a straight line A B. Water would rise to this line in a tube sunk to the porous stratum, and a flowing well would be possible wherever the surface of the ground lies below this line.

Actual conditions may be much modified from those represented in Fig. 4, as where the water is prevented from flowing out at B by reason of an increased density of the stratum or by the stratum becoming thinner. The effect in causing the water to exert an

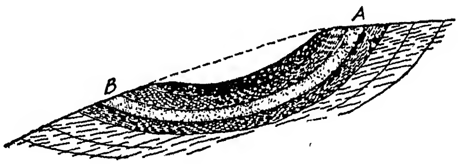


Fig. 4. Dip in Water-Bearing Stratum.

upward pressure is, however, the same. The water-bearing stratum is most often a porous sandstone, although artesian water is also obtained from limestone and in many places from extensive strata of loose uncemented material.

The overlying impervious strata usually consist of clays and shales, these being practically impervious except where fissured. Probably some leakage always takes place through such strata; and many instances are known of large springs which occur at points where the overlying stratum is broken as noted in the preceding section. Except in the case of very limited areas, the capacity of an artesian source as a whole is a question of little importance where it is to be used only for water-supply purposes in towns widely separated; for the total amount of water capable of being drawn from porous rock strata, often hundreds of feet thick and having an outcrop of hundreds or thousands of square miles, is ordinarily very great as compared to any possible demands for such purposes. But wells sunk to tap an artesian stratum must not be placed too close together else they will interfere with one another and the yield per well will be reduced.

25. Predictions Concerning Artesian Wells. The question of the existence of water-bearing strata at any point, their character and depth, and the location of outcrops, is a geological one;

and where full information on this point has not been gained by the sinking of wells or by borings, a geologist familiar with the region in question should be consulted. Much money has often been wasted in fruitless attempts to obtain water in areas and at depths where none could be expected, and frequently such work has been carried on contrary to the advice of experts.

In the construction of wells it is important to preserve samples of the borings, as it is largely through these that a knowledge of the geology of the region is acquired. Chemical analyses of the water are also a valuable aid in identifying strata.

QUALITY OF GROUND WATER SUPPLIES.

26. The quality of ground waters is, in general, quite different from that of surface waters. By percolating through the ground, practically all suspended matter is filtered out, and ground waters are usually clear and sparkling. At the same time this very filtration process causes the water to dissolve more of mineral substances, and the result is that ground waters usually contain much more mineral matter than surface waters. In a limestone country the ground water will be hard, as it will contain lime, and where the soil contains alkali the water will be changed with it. Water that contains little beside lime is not especially objectionable for drinking purposes, but for most other purposes it is more or less expensive and troublesome. An alkali water may be quite unusable.

As regards disease organisms a ground water is likely to be quite free on account of the filtering action of the soil. In the case of private wells, often located near outhouses, pollution is much more likely to occur than in public supplies where any source of pollution must be quite remote.

The temperature, odor and taste of ground waters are generally much more satisfactory than of surface waters. Ground waters constitute a most valuable source of supply for small cities and towns, and where such a supply can be had it should almost always be chosen in preference to a surface water.

CONSTRUCTION OF WORKS.

Before passing on to the details of waterworks construction it will be of assistance to obtain a general view of the subject, and to that end we will here briefly outline the various general features which go to make up a waterworks system.

27. Classification. The various constructive features of a water supply system may be divided into three groups—works for the collection of water; works for the conveyance and distribution of water; works for the purification of water.

28. Works for the Collection of Water. These are divided according to the nature of the source into: (A) Works for taking water from large streams or natural lakes; (B) Works for the collection of ground water; (C) Works for the collection of water from small streams by means of impounding reservoirs.

(A). Works for taking water from large streams or lakes vary in character from a simple cast-iron pipe extending a short distance from shore, to the expensive tunnels and cribs of some of the large cities on the Great Lakes. The location of these works is determined very largely with respect to the quality of the water obtainable. Wherever, as is often the case, it is desired to draw a supply from a lake which at the same time receives sewage from the city, the question is one involving difficulties.

(B). Works for the collection of ground water consist of various forms of shallow wells, artesian wells, filter galleries, etc. The location of works of this class is determined, primarily, by the location of the water-bearing strata. If these are extensive, it will usually be convenient and economical to place the wells at relatively low elevations in order that the water may readily be reached by pumps, or perhaps in order that a flowing well may be secured. In the case of shallow wells the location is often affected by the possibility of local contamination, an element usually absent in the case of deep wells.

(C). Water collected in impounding reservoirs from streams of comparatively small watersheds depends for its good quality chiefly upon the scarcity of population upon the watershed. Suitable areas are therefore more likely to be found in the more rugged parts of the country and at the higher elevations, and usually at considerable distances, sometimes as great as 50 or 75 miles, from the population to be served. The location of such impounding reservoirs is also largely dependent upon questions of construction, such as the location of the dam, length and cost of aqueduct or conduit, and, what is of great economic importance, whether the water can be conveyed and distributed entirely or partly by gravity.

29. Works for the Distribution of Water. These include aqueducts and conduits for conveying water from a distant source, pumps and pumping stations, local reservoirs for equalizing the flow or for storage, and the pipes for distributing to the consumers. Conduits may be open channels, masonry conduits, or pressure conduits, such as pipes of wood, iron, or steel, and sometimes tunnels. The form is determined chiefly by considerations of cost. Pumps are used in a great variety of forms and situations, and may be operated by steam, gas, electricity, wind, or by hydraulic power. There are deep-well pumps for drawing water from depths not reached by suction, low-lift pumps for raising water from a river into settling basins or on to filters, or from wells into a low reservoir; and high-lift pumps for forcing the main supply into the distributing pipes or into an elevated distributing reservoir. Local reservoirs are used for receiving water from long conduits and regulating the flow in the distributing system, for equalizing the flow and pressure in pumping systems, and as settling reservoirs. The pipe system includes distributing mains, fire hydrants, service pipes, shut-off valves, regulating valves, etc.

30. Works for the Purification of Water. These vary in kind according to the nature of the impurities to be removed. Thus in the case of surface waters the sediment, bacteria, etc., are removed more or less completely by settling basins and various forms of filters. In the case of ground waters iron may be removed by aeration and filtration; hardness by chemical precipitation, etc. In these ways waters otherwise very undesirable can be greatly improved or made entirely satisfactory, but of course at a considerable expenditure of money. It will often happen, therefore, that a source of good quality but expensive will need to be compared with another poor in quality but capable of being made fairly comparable with the other at no greater total cost. Not infrequently the possibility of the future deterioration of a surface supply and the consequent necessity for artificial purification must also be considered.

RIVER AND LAKE INTAKES.

In drawing a water supply from a large river or lake a pipe or tunnel must extend from the pumping works out some distance from shore and the construction of such pipe line or tunnel often involves some very difficult work.

31. River Intakes. The location of the point of intake must be selected with reference to (1) the quality of water, and (2) the cost of construction and maintenance of the works connected therewith. The point of intake should be free from local sources of pollution and should therefore be located above all sewer outfalls of the town in question. In the case of tidal streams, sewage-polluted water may be carried long distances above the respective outfalls at flood tide, and before selecting the location careful study should be made of this question by means of floats and by examinations of the water at various seasons of the year. The location of the intake must also be determined with special reference to the lowest water stage.

The form of construction to be used depends upon the character of the stream in question, especially whether the difference between low and high water level is small or great.

If the water level vary only a small amount, as in the case of streams near dams or near a lake or ocean, the water may usually be taken from near the shore, the end of the intake pipe being supported on a small foundation of concrete, or on a wooden crib, or by a masonry retaining wall.

The intake pipes, usually of cast iron, may lead directly to the pumps, thus acting as suction pipes, or to a gate chamber and pump well. In the latter case the suction pipes of the pumps lead from this pump well. Gratings of cast iron or wood, with large openings, are usually placed at the entrance to the intake to prevent the admission of large objects, while fish screens of copper are inserted in the gate house or placed over the ends of the suction pipes.

If there is a large fluctuation of water level considerably more work is involved. It is usually necessary to extend the intake pipe a considerable distance from the banks of the stream in order to reach a suitable location at low water. Furthermore, pumps cannot lift by suction more than about 20 feet in practice, hence in order to enable the pumps to reach the water at the lowest stage, it is often necessary to place them in a deep pump pit much below high water level. The construction of a water-tight pit for this purpose is then an important feature of the works.

Another form of construction at the end of the intake is a masonry tower extending above high water and containing ports and sluice-

gate similar in form to those used in reservoirs. To provide stability against ice and drift the tower is built similar to a bridge pier in form, the inlet ports being placed along the sides. The outer end of the intake pipe is usually protected by a simple timber crib supporting the end of the pipe 2 or 3 feet above the river bottom, and held in place and protected from scour by broken stone. A coarse screen or grating is ordinarily placed over that compartment of the crib containing the intake pipe. It is desirable to have the total area of the openings of this grating 2 or 3 times that of the pipe itself in order to keep the entrance velocity low. Sometimes in order to strain out the sediment the crib is entirely filled with broken stone and sand to form a filter crib. Such intake towers are used at St. Louis and at Cincinnati and tunnels connect with the tower through which the water is conveyed to the pumps.

The tower has the advantage over the crib construction in permanence and reliability. For these reasons this form of construction is to be commended, but it is much more expensive than the crib construction and is therefore suited only for the larger and more important works.

From the crib or inlet tower the intake pipe or tunnel usually runs to a screening chamber or pump well and from this chamber suction pipes lead direct to the pumps.

Fig. 5 illustrates a good design for small works. Here the water flows by gravity to the wet well made of boiler steel and con-

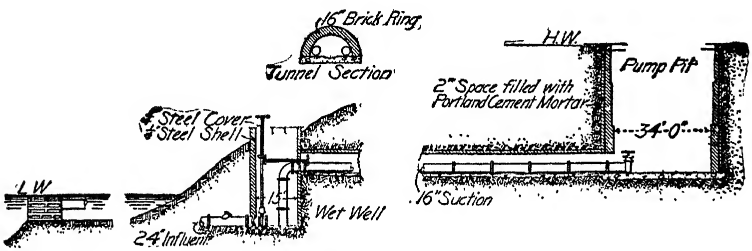


Fig. 5 Intake on the Ohio River

structed below high water line. From this well the water is drawn by suction pipes attached directly to pumps in the pump pit. The suction pipe is placed in a tunnel through which access may be had in time of high water to the valves in the wet well. The size of

intake pipe and suction pipe should be such that the velocity of the water in them will not exceed $1\frac{1}{2}$ to 2 feet per second.

32. Lake Intakes. The location of a lake intake in such a position as to obtain at all times water of the best quality, and to fulfill the requirements of safety against interruption, is a question requiring very careful study. In a lake unpolluted by sewage some of the things to be investigated are—the location of the mouths of streams and the sediment carried by them; the character of the lake bottom; the direction of wind and currents and their effects in stirring up the mud on the lake bottom and in conveying sediment from point to point.

The intake should if practicable be located at a sufficient depth to be free from any considerable wave action, both to secure a greater stability and to avoid the effect of the disturbance of the sediment by the waves. Even in small ponds the wind stirs up the water to a depth of 15 or 20 feet, so that this may be taken as about the minimum depth. A greater depth is desirable if the water is not too stagnant, since the water becomes rapidly cooler below this point. In large lakes the wave action extends to much greater depths and the intake should be extended accordingly to depths of 40 or 50 feet.

Most of the cities along the Great Lakes dispose of their sewage by running it directly into the lake at the most convenient point; and for those places that draw their water supply from the same body of water the most difficult part of the intake problem is to exclude their own sewage. As the cities grow, the intakes are pushed farther and farther out, but usually not until the necessity of the step is brought home by increased mortality from typhoid fever; and, however carefully this matter is followed up, the quality of the water taken from such sources must always be looked upon with suspicion. In Chicago the length of intake has gradually increased to 4 miles. In Milwaukee it is $1\frac{1}{2}$ miles, while the new intake at Cleveland is about 5 miles long.

Whether the conduit should be a pipe line or a tunnel depends upon the cost of construction and the relative reliability of the two forms. In small works the cost of a tunnel would be prohibitory, while in the case of a very large intake a tunnel may be the cheaper. Again, a pipe-line, unless sunk very deep, is subject to disturbances near the shore end by ice action, wreckage, and scour from storms.

Submerged-pipe intakes are usually laid by the aid of divers, although other methods have been used. The pipe is preferably laid in a dredged trench, at least as far out as wave-action is to be feared, and should be covered generally to a depth of 3 or 4 feet. Near the shore end the covering should be considerably deeper than this. Various methods of laying submerged pipe are described later.

Most lake intakes are protected at their ends by submerged crib work of timber partly filled with stone, the end of the pipe being raised 6 or 8 feet above the lake bottom to prevent the entrance of sand. At some of the larger ones, as at Chicago and Cleveland, large inlet cribs or towers built above the water surface are used, similar to river inlet towers.

The greatest difficulty met with in operating lake intakes is due to the clogging of the ports by anchor ice. This consists of needles and thin scales of ice which form in moving water and which are of such small size that they are readily carried below the surface by comparatively weak currents. They cease to form after the body of water has become frozen over. On coming in contact with submerged objects these particles of ice adhere and soon form large masses difficult to dislodge. Anchor ice has given much trouble at lake intakes both at the exposed cribs and at the shallower submerged ones. It is removed in various ways. Compressed air discharged near the port has been effective in some submerged intakes. Steam, water from hose, chains drawn back and forth through the ports, and pike poles, are some of the other means used. As tending to obviate difficulty with anchor ice all crib openings or port holes should have a large area so that the velocity of flow through them will not be more than 3 or 4 inches per second.

WORKS FOR THE COLLECTION OF GROUND WATER.

33. Collection of Water from Springs. The chief objects to be accomplished in the construction of works of the kind here considered are the protection of the water from pollution and the spring from injury through clogging or otherwise, the furnishing of a convenient chamber from which the conduit pipes may lead, and, in some cases, the enlargement of the yield by suitable forms of construction.

If a supply sufficient at all times for the demand can be obtained from one or more large springs, each one should have its separate

basin from which the water may be conducted to a common main. The simplest form of works consists of a small masonry well or basin surrounding the spring and from which the conduit pipe leads. To prevent a growth of vegetable organisms and consequent deterioration of the water, such basin should always be covered so as to exclude the light. For a small spring, a circular well covered with a stone cap cemented in place and provided with a manhole is a simple and effective arrangement. For larger springs a masonry vault covered with 2 or 3 feet of earth is preferable. If the spring is located on a steep hillside, the collecting chamber is conveniently constructed in the form of a horizontal gallery built into the hill, access to which is had through a door or manhole.

Mineral and other springs occurring in public places usually have open basins, and opportunities are offered in the walls and parapets for ornamentation.

If the natural yield of a spring is insufficient, it will sometimes be possible to increase it. The proper form of collecting works to accomplish this depends upon the character of the spring. If the water appears at the upper surface of a stratum of impervious material overlaid by the water-bearing deposit, frequently in the form of several small springs, instead of dealing with each one individually it will often be better to construct a long collecting gallery running parallel to the outcrop and leading to a central collecting chamber which can be made similar in form to that for a large spring. This gallery, which may be made similar to that shown in Fig. 12, should be built deep enough to rest upon the impervious material, and thus to collect all the underground flowage as well as that appearing as springs. The total yield may be thus much increased, the increase being relatively greatest during dry weather.

The gallery may be simply a line of drain tile or vitrified pipe laid with open joints at the upper part of the pipe. If large quantities are collected the gallery may be made of brick or stone and large enough to permit of the passage of a man.

Where springs originate in a deep porous stratum such stratum may usually be tapped by wells without much reference to the spring. Use of such wells will generally reduce or entirely stop the flow of the spring.

THE CONSTRUCTION OF WELLS.

34. Principles Governing the Yield of Wells. If a well, either large or small, be sunk into a body of water-bearing material the water will run into such well, and if no pumping is done the water will, after a time, reach a level in the well the same as the level of the water in the surrounding soil. Fig. 6 represents a section through such a well. The dotted line A B represents the level of the water in the ground and in the well. Now if water is pumped from this well the level of the water therein will be lowered and as a consequence water will tend to flow into it from the surrounding ground and the surface of the ground water will assume some such shape as shown by the full line C D E F. The amount which the water surface is lowered decreases rapidly as we get farther from the well, until at some point more or less remote there is no sensible effect. The area within which the level is appreciably lowered is called the *circle of influence*. If the pumping is continued the level will be more and more lowered until it is so low that water will run into the well as rapidly as it is pumped out, after which no further change will take place. If the pumping ceases the well will gradually fill up to the original level.

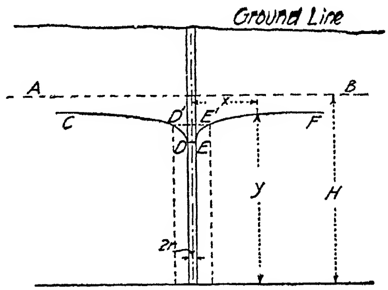


Fig. 6. Well Sunk in Ground Water.

Where the water flows under pressure, as in a porous stratum overlaid by an impervious one, the flow into a well is not accompanied by a change of *level* in the surface water, but the curve of *pressures* is of a form similar to the water surface in the case already treated.

The principles underlying the yield of wells have been investigated both theoretically and practically, but the subject is too difficult to be discussed in detail here. There are certain general principles, however, that are very important and which aid greatly to a clear understanding of the behavior of a set of wells under varying conditions. These may be stated as follows:

Having given a sand or gravel stratum of at least several feet in thickness in which water is flowing at some appreciable slope,

such as 5 or 10 feet per mile, and a well is sunk into this stratum to a considerable depth, the yield of such a well when pumped from continually will follow approximately the following laws:

(1) The yield will be proportional to the distance the water level is lowered in the well below its normal level.

(2) The yield will be proportional to the thickness of the water-bearing stratum.

(3) For the same amount of lowering of the water the yield will be a little greater the larger the well, but the difference is not great except in case of very deep wells of small diameter in which the upward velocity of flow through the well is greater than 2 or 3 feet per second. A 10-foot well will yield only about 50 per cent more than a 6-inch well.

(4) For the same amount of lowering of the water the yield will be much greater in coarse material than in fine.

The following table will serve to give a rough idea of what may be expected from a single well sunk at least half way through a water-bearing stratum of various grades of material.

TABLE 9.

Approximate Yield of a 6-Inch Well When Sunk Into Water-Bearing Material 10 Feet Thick and When the Water Level Is Lowered One Foot by Continuous Pumping.

Material.	Yield in gallons per day.
Fine Sand	4,000
Medium Sand	30,000
Coarse Sand	80,000
Fine Gravel, free of Sand	500,000 or more.

For other thicknesses of material and other amounts of lowering the yield can be obtained by the law of proportion as stated above. The great increase in yield due to increasing coarseness of material is very marked and shows that for this very reason it is very difficult to make close predictions as to yield. Larger wells will give slightly better results.

Example. A well is sunk into a water-bearing stratum consisting of medium size sand to a depth of 30 feet below water level. What will be the yield if the water therein is pumped down 5 feet below its normal level?

By Table 9 the yield would be about 30,000 gallons per day for a 10-foot stratum and one foot of lowering. Hence for a 30-foot

stratum and 5 feet of lowering the yield will be about $3 \times 5 = 15$ times as much, or $15 \times 30{,}000 = 450{,}000$ gallons per day.

If two or more wells penetrating to the same stratum are placed near together and simultaneously operated, the total yield will be relatively much less than the yield of a single well pumped to the same level. This mutual interference of wells depends in amount upon the size and spacing of the wells, upon the radius of the circle of influence of the wells when operated singly, and upon the depth to which the water is lowered by pumping.

The amount of this interference depends mainly upon the distance the wells are apart. It also depends upon the amount the water is lowered by pumping, and upon the general capacity of the stratum. If the water is lowered a considerable amount, such as 10 feet, the wells should be placed 200 to 400 feet apart in order that the interference be not too great. A small spacing like 25 to 50 feet will give an interference of a large amount,—often as great as 50 per cent in the case of 3 or more wells. That is to say, if 4 wells are placed 50 feet apart the total yield is not likely to be more than 50 per cent of the yield if these 4 wells were placed 300 or 400 feet apart.

Where it can be done, the best way to determine the capacity of wells is by actual tests conducted for a sufficient length of time to bring about a condition of equilibrium in the flow, but unless this condition is approximately fulfilled such tests are apt to be very deceptive. With a flat slope to the ground water a test may be carried on for weeks and even months, and the circle of influence will still continue to widen, resulting in a gradually decreasing yield. It may thus require years of operation to bring the conditions to a final state of equilibrium.

In the case of deep wells sunk into rock strata it is impossible to make an analysis of the conditions so as to be able to predict the yield. A pumping test is a necessity, but in this case also a very useful principle to remember is that of proportionality of flow to the lowering of the water level in the well. Thus if by pumping the level down 10 feet we get 200,000 gallons per day we may say with great certainty that the yield will be about 400,000 gallons if the water is pumped down 20 feet. In all cases this lowering of the water is to be measured from the level to which it rises when

no water is pumped. In a flowing artesian well to get this level it is necessary to extend the casing above the ground as far as the water will rise, or to cap the well and determine this level by a pressure gauge.

35. Large Open Wells. As already explained, the yield of a well that is constantly pumped from is not much affected by its size. For other reasons, however, large wells are often advantageous.

The large well possesses a great advantage over the small well in its storage capacity. If the pumping is carried on at a variable rate, it thus acts to increase greatly the real capacity of the large well over that of a series of small tube wells. Furthermore, in the operation of the pumps there are many advantages in being able to get the entire supply from a single well, or from two or three large wells close together, chief among which is the avoidance of long suction pipes. The large well is also of great advantage where it becomes necessary to lower the pumps, as it permits the use of a more economical form of pumping machinery.

Trouble is often experienced in the small wells through clogging and the entrance of fine sand. This is largely avoided in the large well, as the entrance velocity of the water is very small. Opportunity is also given for the settling of fine material.

The chief disadvantage of the large well is in its great cost compared to the tube well for like yields. This disadvantage increases rapidly as the depth increases, and where it may be economy to construct a large well to a certain depth to serve as a pump pit it will usually be cheaper to develop the yield by sinking tube wells from the bottom, or by driving galleries therefrom, than by further sinking.

Large wells for waterworks are constructed of diameters of 10 feet or less to as great as 100 feet, 30 to 50 feet being the most common size. The minimum depth of a well is determined by the depth necessary to reach and penetrate for a short distance the water-bearing stratum, allowing a margin for dry seasons.

In the construction of a large well large quantities of water will be met with, and adequate means of handling it must be provided. As the water level must be kept at the lowest level of the excavation, the maximum pumpage will be considerably more than the future capacity of the well. For moderate depths the excavation can be carried on with no other aid than sheet piling. If the well

is of large diameter, an annular trench is usually first excavated and the curb or lining built therein, after which the interior core is removed. This method enables the sheet piling to be readily braced. A method adapted to smaller wells is to drive the sheet

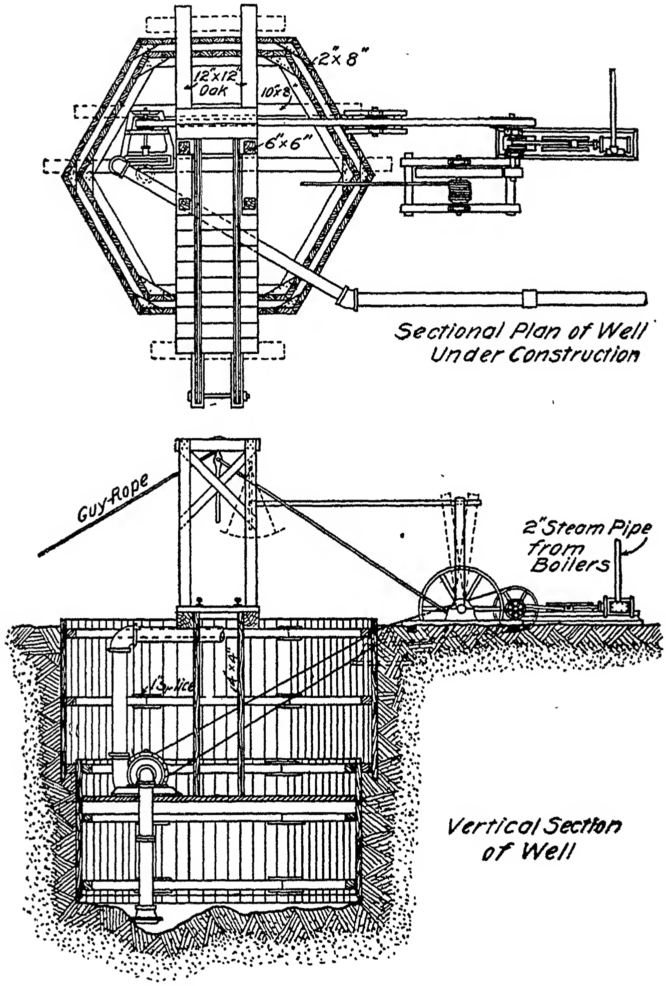


Fig. 7. Large Well under Construction.

piling outside of a series of wooden frames or ribs, and to excavate the entire well at once. The ribs are built in place as the excavation proceeds. This method is illustrated in Fig. 7.

For wells of considerable depth sunk in soft material, the curb may be started on a shoe of iron or wood, and the excavation and the construction of the curb carried on simultaneously, the curb sinking from its own weight. The material may be either excavated in the ordinary way, or by the use of compressed air, or dredged out without attempting to keep out the water, the method used depending upon depth of well, quantity of water, and character of the material. Where the friction becomes too great to sink the first curb the desired distance, a second curb with shoe may be sunk inside the former. In Fig. 8 are illustrated two forms of shoes used in sinking wells. These are both constructed mainly of wood. To strengthen such curbs iron rods should extend from the shoe well up into the masonry. For large wells, pump pits, etc., heavy iron shoes are often employed, and occasionally a pneumatic caisson is found necessary.

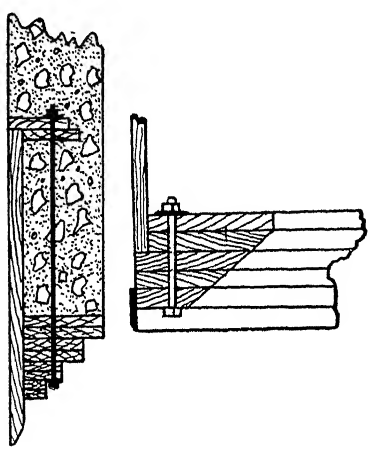


Fig. 8. Shoes for Sinking Well Curbs.

The lining or curb usually consists of a circular wall of brick or concrete masonry of a thickness varying with diameter and depth of the well, but ranging ordinarily from 2 to 5 feet. Dry rubble may be used for the lower portion, but the upper portion should be of concrete.

All wells should be covered to exclude the light and to prevent pollution of the water. The cover is usually made of wood, which for large wells may be conveniently made of a conical form and supported by a light wooden truss, or by rafters resting against the wall.

36. Shallow Tubular or Driven Wells. Shallow tubular wells, or wells of small diameter, also called *driven* wells, are sunk in various ways, depending upon the size and depth of well and nature of the material encountered. To furnish large quantities of water it usually requires a number of wells, and in addition to the question of sinking, questions of arrangement, spacing, con-

necting and operation are important. We will here consider only the methods of sinking wells in earth or soft strata.

As regards methods of sinking there are two principal kinds of wells—the closed-end well or driven well proper, and the open-end well.

The Closed-end or Driven Well. In this form the well tube consists of a wrought-iron tube from 1 to 4 inches in diameter, closed and pointed at one end, and perforated for some distance therefrom. The tube thus prepared is driven into the ground by a wooden maul or block until it penetrates the water-bearing stratum. The upper end is then connected to a pump and the well is complete. Where the material penetrated is sand the perforated portion is covered with wire gauze of a fineness depending upon the fineness of the sand. To prevent injuring the gauze and clogging the perforations, the pointed end is usually made larger than the tube, or the gauze may be covered by a perforated jacket.

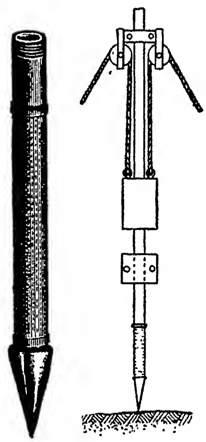


Fig. 9. Well Point and Driving Rig.

Fig. 9 shows a common form of well point and the method of driving wells by means of a weight operated by two men. The tube may also be driven by a wooden block operated by a pile driver or other convenient means. Such a well is adapted for use in soft ground or sand up to a depth of about 75 feet, and in places where the water is thinly distributed.

Open-end Wells. For use in hard ground and for the larger sizes the open-end tube is better adapted. This is sunk by removing the material from the interior, and at the same time driving the tube as in the other case. A very common method of sinking is by means of the water jet. In this process a strong stream of water is forced through a small pipe inserted in the well tube, the water escaping in one or more jets near the end of the pipe. At the same time the pipe, which is provided with a chisel edge, is churned up and down to loosen the material, which is then carried to the surface by the water in the annular space between the pipe and tube. If the material is hard or the well deep, a steel cutting edge may be screwed on to the end of the well tube.

With the open-end well the lower portion may be merely perforated with small holes in case the material is coarse or gravelly, or if sand is met with, the holes may be covered with brass gauze. Instead, however, of using a gauze it is common with this style of well to sink a solid tube, insert a special strainer of suitable length, and then withdraw the tube nearly to the top of the strainer.

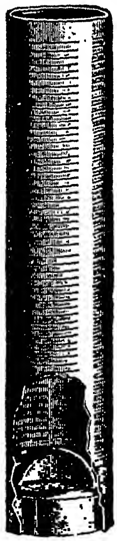


Fig. 10. Cook Well Strainer.

Fig. 10 illustrates a commonly used form of strainer known as the Cook strainer. It is made of brass tubing and provided with very narrow, slotted holes, which are much wider on the interior than on the exterior, an arrangement intended to prevent clogging.

Small tubular wells are usually arranged in one or two rows alongside a suction pipe and connected thereto by short branches. The smaller sizes are connected directly to the branch, the well tube acting also as a suction pipe, but with the larger sizes a separate suction pipe is ordinarily employed. In the former case, to avoid the entrance of air, it is necessary that the perforated portion of the pipe be always under water, and to insure this being the case it should be kept below the limit of suction. With the latter arrangement there are no such limitations to the position of the perforated well casing.

In order to enable the pumps to draw as much water as possible from the wells the pumps and suction main should be placed as

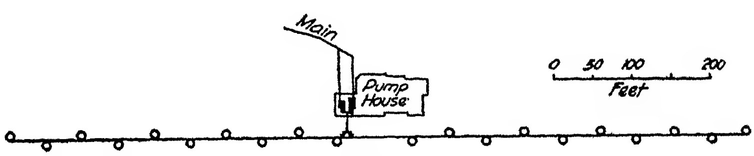


Fig. 11. Typical Arrangement of Wells.

deeply in the ground as practicable. A typical arrangement of wells is shown in Fig 11. In this plan the wells are 6-inch wells and are spaced 50 feet apart and are 35 to 50 feet deep.

The maximum amount of water obtainable from a given number of wells would be when they are spaced far enough apart so that their circles of influence will not overlap, but on account of cost of piping, and loss of head by friction, this would not be the most economical spacing. While it is impossible to give figures which would be of general application, it may be stated that from 25 to 100 feet is about the range for economical spacing of shallow wells. With very deep or artesian wells the spacing becomes still greater. Spacing less than 25 feet has quite often been used, but with doubtful economy.

Each well should be connected to the suction main by means of a short branch in which should be placed a gate valve, so that any well can be shut off at any time. The main suction pipe is usually made of flanged pipe, as this enables air-tight joints to be more readily made, although ordinary bell-and-spigot pipe with lead joints has been successfully used.

The greatest care must be taken in every part to make the work air tight, and to secure this it should be thoroughly tested in sections by means of compressed air. All valves should be carefully tested for air tightness, and all screw connections thoroughly fitted. In spite of the most careful construction, air will usually accumulate to some extent, and to eliminate it many plants are provided with air separators placed on the suction main near the pump. The simplest form consists of a large drum of steel placed on the suction pipe near the pumps through which the water passes at a slow velocity. A vacuum pump is attached to this drum.

Where sand is drawn up with the water it may be got rid of by passing the water at a slow velocity through a large drum or box inserted in the suction pipe and provided with suitable handholes for cleaning.

37. Deep and Artesian Wells. Where the depth exceeds 75 to 100 feet the small driven well is no longer practicable. Methods of sinking deep wells are in many respects different from those already described, and matters of spacing, pipe friction, arrangement of connections, etc., are much more important than in the shallow-well plant. Well boring is an art by itself, and the execution of any deep-well project should usually be put into the hands of some reliable well-drilling concern. The variety of ingenious tools and appliances in use for overcoming all kinds of difficulties and for penetrating all

sorts of strata is very great, and it is possible to give here but a very general description of some of the methods of sinking in use.

In soft material it is necessary to case the well the entire depth, and on account of the difficulty of getting the casing down to great depths this operation becomes the chief feature of the construction. For depths up to 200 or 300 feet the ordinary well-drilling outfit can be used, and the casing driven close after the drill. When the casing can be driven no farther a smaller size is inserted and the sinking continued with a smaller drill, and so on until the well is sunk as far as desirable or possible. The material excavated is brought to the surface by means of a sand bucket, or by the water jet as previously described, the water being conducted to the end of the drill through hollow drill rods. By the latter method the hole is kept clean and a more rapid progress made.

The friction against the casing is greatly lessened, and the depth attainable much increased by the use of the revolving process. In this the lower end of the casing is provided with a toothed cutting shoe of hard steel of slightly greater diameter than the pipe, and the upper end is connected by means of a swivel to a water pipe through which water is forced by suitable pumps. The well is bored by turning the pipe, and the loosened material is carried to the surface by the water which passes down inside the casing and up on the outside between casing and soil. This process is very common in sinking artesian wells in the alluvial basins of California. It is very rapid, a rate of sinking as high as 20 or 30 feet per hour for depths of 1,000 feet having been attained.

It is essential to have a good length of strainer in the porous stratum. This is usually inserted after the desired depth has been reached, and the casing is then pulled up to the top of the strainer. By special devices it can, however, be attached to the end of the well casing and sunk with it.

A drilling outfit for deep wells is very similar to the ordinary familiar outfit for shallow wells worked by horse-power. A string of tools consists essentially of a steel bit, an auger-stem into which the bit is screwed, a pair of links or "jars" connecting the auger stem with another bar, called a sinker bar, and finally the rope cable which supports the apparatus and which passes over a pulley at the top of a derrick and then down to a winding drum. Just above

the drum the cable is attached, by means of an adjusting or "temper" screw, to a large walking beam operated by a steam engine. As the work progresses the drill is lowerd by the temper screw. By means of the jars an upward blow may be struck to dislodge a jammed drill. Many ingenious tools are employed for recovering lost tools, cutting up and removing pipe, and carrying on the various operations involved.

Wells in soft material must be cased throughout. When bored in rock it is necessary to case the well at least through the soft upper strata to prevent caving. Casing is also desirable for the purpose of excluding surface water, to which end it should extend well into the solid stratum below. Where artesian conditions exist and the water will eventually stand higher in the well than the adjacent ground water, the casing must extend into and make a tight joint with the impervious stratum, otherwise water will escape into the ground above.

Ordinary artesian well casing is made of light-weight wrought-iron lap-welded pipe. For pipe which is to be driven the standard wrought-iron pipe is ordinarily used, but for heavy driving extra strong pipe is necessary. Joints of drive pipe should be made so that the ends of the tubing are in contact when screwed up. The life of a good heavy pipe is ordinarily very great, but cases have occurred where the pipe has been rapidly corroded, due to the presence of excessive amounts of carbonic acid.

The cost of sinking wells will of course vary greatly according to locality, nature of strata, and depth and size of well. For wells 6 to 8 inches in diameter and sunk in ordinary rock the cost per foot, not including casing, will usually range from $2.00 to $3.00 for depths of 500 feet, up to $3.00 to $5.00 for depths of 2,000 feet. For smaller sizes the cost will be somewhat less, especially for the shallow depths.

38. Connections for Deep Wells. The economical spacing for deep wells will be much greater than for shallow wells. It will likewise pay to spend more money in lowering the flow line by making deep connections, thus decreasing the number of wells and increasing the spacing. Generally speaking a spacing of from 400 to 800 feet will be found desirable.

On account of the relatively great cost of deep wells it will often be found economical to so arrange the pumps and connections that

a considerable lowering of the water level below the ground surface may be obtained. This is generally accomplished by connecting all the wells to a single pump or set of pumps, placed at a considerable depth below the surface. Where the connections are very deep tunneling may have to be resorted to. Another common method of drawing water from deep wells in the case of small plants is by the use of a separate deep-well pump for each well. This method is applicable to any depth, but involves the use of uneconomical types of machinery. The air lift is another form suited to this work.

39. Yield of Artesian Wells. In making estimates regarding flow it is important to bear in mind that it requires a considerable length of time to determine with certainty the adequacy of the supply, and furthermore that the sinking of wells by other interests, even though at considerable distances, may very seriously affect the yield. Where conditions are sufficiently favorable for works of some magnitude the yield per well under a moderate head ranges from about 150,000 gallons per day to 800,000 gallons, or even more. With yields of less than 100,000 gallons per day, works for developing large quantities become very expensive, relatively more expensive than for small quantities, since with a large number of wells there is much greater interference. Often a well or set of wells will show a gradual falling off in capacity. The chief cause of a decrease in the yield of a well is the influence of other wells sunk in the vicinity. Where large numbers of wells are sunk in the same region this effect may be very serious, as in some cases where it has reduced the pressure of flowing wells from 75 or 100 feet down to nothing.

40. Galleries and Horizontal Wells. Where ground water can be reached at moderate depths it is sometimes intercepted by galleries constructed across the line of flow. If these are placed at a sufficient depth they will enable the entire flow of the ground water to be intercepted. In form a gallery may consist merely of an open ditch which leads the water away, or it may be a closed conduit of masonry, wood, iron, or vitrified clay pipe, provided with numerous small openings to allow the entrance of the water. Unless constantly submerged, wood should not be used. Masonry and vitrified pipe are preferable to iron, as these materials are uninjured by exposure to water. If galleries are not covered, excessive vegetable growth is apt to occur which may injure the quality of the

water. Fig. 12 illustrates a form of gallery of concrete built in a water-bearing gravel.

Galleries for collecting ground water are occasionally tunneled in solid rock. This may happen along a side hill where an outcropping porous stratum overlies an impervious one and it is desired to develop the flow by running a tunnel along the hill near the bottom of the porous stratum; or it may occur where a steeply inclined artesian stratum can be more readily reached in this way than by vertical wells. Tunnels or galleries are also sometimes run from the bottom of large wells for the purpose of increasing the yield. This method of increasing the flow is advantageous where it is necessary to lower the pumps and to concentrate the flow in a single well.

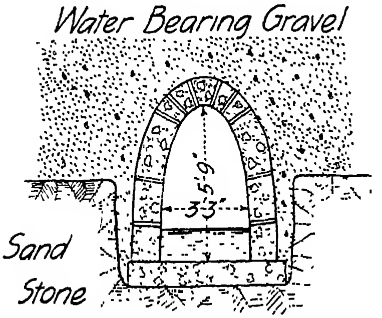


Fig. 12. Concrete Gallery.

Horizontal or push wells are tubular wells pushed approximately horizontally into a water-bearing stratum, or under the bed of a lake or stream. They are forced into the ground from a trench by means of jacks braced against the opposite side. These wells have been most successful when extended out under a lake or river.

Another method of utilizing a river bottom as a natural filter is to construct a wooden crib in an excavation in the bed of the stream, fill it with gravel and then cover the structure with 3 or 4 feet of sand up even with the river bottom. The suction pipe then leads from the crib to the pumps. This form of construction is well adapted to sandy-bottom streams with swift currents and has proved a very efficient way of clarifying muddy river waters.

Wells and galleries are often constructed near streams for the purpose of getting all or a portion of the supply therefrom. The success of such works depends much upon the character of the river bottom. Even when the lower strata are porous, the river, if a silt bearing one, may have a nearly impervious bottom and the natural filter will only become more clogged by use, necessitating perhaps the abandonment of the collecting works. Such failures have occurred in some instances. With a sandy river bottom kept

clean by the scouring action of the floods, and with a porous substratum, works of this kind will give good results. To secure good filtration the works should be located at least 50 feet and preferably a greater distance from the stream.

The yield of a series of wells or of a gallery collecting filtered surface water will be, as in the case previously discussed, proportional to the lowering of the water level, and will be nearly proportional to the length of the line of works. In gallons per day per 100 feet of gallery, the yield from various successful works varies from 30,000 to 1,000,000 or more, which is about the same as is obtained from lines of wells.

RESERVOIRS AND DAMS.

Impounding Reservoirs.

41. Capacity. When the minimum flow of a stream is less than the daily demand of water it is necessary to store up the excess flow during the rainy season in large reservoirs called *impounding reservoirs.* The deficiency in the supply can then be made up by drawing from the reservoir. In this way the entire flow of a stream for a year or more may be stored and drawn off as wanted and streams that run dry at certain times may be made to supply quite a large population. Impounding reservoirs are made by constructing a dam across the valley in question, but natural lakes or ponds can often be utilized as reservoirs by building suitable works at this outlet.

In calculating the proper size of a reservoir we must consider (1) the yield of the source for successive intervals of time; and (2) the demand for all purposes for like intervals of time. The yield of the source of supply has been previously discussed. The demand to be considered includes not only the consumption for the city in question, but also the loss of water by evaporation from the area of the reservoir itself, also loss from leakage and percolation, and often the necessary withdrawals to satisfy the demands of riparian owners below.

The amount of leakage through the dam will usually be very small, but with certain forms of construction may be large. The quantity of water necessary to satisfy the demands of the riparian owners below the reservoir is often an exceedingly difficult matter to determine, and usually becomes a question for the courts to settle.

Practice differs greatly in different States, and in many of the Western States the water belongs to the State to dispose of as it sees fit. It is often expedient to buy up all rights and to utilize whenever necessary the entire flow of a stream, or to fix by contract the amount which will be allowed to flow.

The capacity of the reservoir must be based on the supplying capacity of the stream during the dryest year. The probable yield of the stream for each month of such a year should be estimated and recorded. Then likewise the monthly demand for the city in question and whatever allowance, if any, should be made for the use of riparian owners below.

Then for all months in which the demand is greater than the flow subtract the latter from the former; this will give the deficiency for each month. Add all deficiencies together and the result will be the total deficiency which must be made up from the reservoir and therefore is the required capacity of the reservoir, provided, however, that the total surplus for the remaining months is at least equal to the deficiency. If not, then the total yearly flow of the stream is not equal to the total demand and additional water must be obtained from some other source.

42. Location. In determining upon the location of a reservoir several elements must be kept in mind. In the first place it is very desirable that it shall be at such an elevation that at least a part of the consumers may be served by gravity alone, and it will be economy to spend a relatively large sum of money for conduits, or otherwise, to secure this advantage. The necessary elevation for this purpose depends upon the required pressure at, and elevation of, the various points of distribution, and the head lost in conducting thence the water.

The most favorable location for a reservoir as regards topography is a point where the valley forms a comparatively broad level area bounded by steep slopes at the sides, and below which the hills approach close together so as to form a good site for a dam. To prevent the escape of water the floor of the reservoir should contain no outcrop of porous strata of any extent which may lead the water away underground, and in the vicinity of the dam or embankment it should be underlain by an impervious stratum at a depth that can be reached by that structure.

After a tentative location has been decided upon, accurate levels must be run to connect the town with the reservoir site, also surveys for conduit lines, and an accurate topographical survey of the area to be flooded and all that may be affected by the reservoir. This survey should include information as to all buildings upon and adjacent to the area in question, nature of the vegetation, location of roads, property lines, etc. At the site proposed for the dam numerous borings must be made extending to a considerable distance above and below the dam as well as on the flanks, and these must be supplemented by test pits so that the nature of the supposed firm stratum can be accurately determined. If a suitable foundation cannot be reached at a reasonable cost, the site may have to be abandoned.

Calculations of storage volumes for different depths can readily be made from the contour map. The areas enclosed by each contour can be measured by a planimeter and the volume between any two successive contours taken as equal to the average of the areas enclosed by the contours, multiplied by the contour interval. The volume up to any given contour having been determined, the necessary height of dam to hold any given quantity of water becomes known.

All vegetation and perishable matter should be removed from the reservoir site, as the decay of such material injures the quality of the water. It is also desirable and of great benefit to the water to remove the top soil to a sufficient depth to include most of the organic matter therein.

As a further protection to the quality of the stored water it is desirable that there be as little area alternately flooded and exposed as possible, in order to limit the growth of vegetation. Shallow places should either be excavated to give a depth of 6 or 8 feet, or partly excavated and partly filled, the slopes being formed at about 3 to 1 and covered with sand or gravel.

43. Maintenance. In maintaining a reservoir so as to preserve the quality of the water and to supply the necessary quantity regularly and certainly requires a considerable degree of care and attention. To keep the quality as good as possible requires first of all that the watershed and reservoir be kept free from organic pollution. To insure that this is the case the city should have sani-

tary supervision over the area in question, and inspection should be regularly made to see that all sanitary requirements are complied with. During seasons of low water, opportunity is offered for removing the vegetation from around the borders of the reservoir.

Careful records should be kept at the reservoir of all matters which may be of any value in subsequent designs for enlargement or for new works. These should include records of rainfall, temperature, height of water in reservoir, amount passing over waste weir, and data pertaining to the quality of the water at different seasons of the year. The maintenance of dams and embankments should call for very little labor. Earthen embankments should be kept neat in appearance with slopes well sodded, or covered with large gravel so as to be permanent. The top of the embankment should of course be maintained at its full height, and the waste weir and the channel below it kept clear and of the designed capacity at all times. Gates and other apparatus should be frequently inspected and kept in thorough repair.

EARTHEN DAMS OR EMBANKMENTS.

44. Kinds of Dams. Dams may be divided according to the material used into five classes; earthen dams, masonry dams, loose-rock dams, wooden dams, and iron or steel dams. These materials are also used in various combinations. The form of dam suitable for a given case depends upon the character of the foundation, the topography of the site, the size and importance of the structure, the degree of imperviousness required, and the cost. Of the above kinds of dams those of masonry and of earth are the ones usually considered.

The earthen embankment is the most common form of dam. It can be built on a variety of foundations; it is commonly the cheapest form, and when well designed and executed is an entirely safe and reliable structure. Where flood waters have to be passed over a dam some other material than earth must be used for at least the portion of the structure subjected to water action. Water flowing over an earthen embankment is inadmissible, many failures having been caused by such occurrence, due to faulty construction. For dams higher than 100 feet or thereabouts few engineers would recommend an earthen structure. If the foundations are suitable, a

masonry dam is in such cases greatly to be preferred. It is more reliable, and with the great pressures occurring it is desirable to have all outlet arrangements built in masonry.

The general requirements of a good foundation for an earthen dam are that an impervious stratum can be reached at a moderate depth, and that the material near the surface is sufficiently compact to support the load. A compact clay or hardpan makes the best foundation. Solid rock is also good if not fissured. Embankments of earth have been successfully constructed on foundations of sand; but in such a case it is important that the sand be fine and of a uniform character, containing no streaks of coarse material which will offer little resistance to the flow of water.

Earthen dams are of a trapezoidal form with top width, side slopes, etc., proportioned according to the material used. Where good material is at hand in sufficient quantities the entire embankment may be made of uniform consistency and all as nearly water tight as possible. Usually, however, it will be more economical and give as good results to put the best material near the upper side of the embankment, changing gradually to the more porous materials towards the lower face. Where good material is scarce, imperviousness is usually obtained by means of a wall or "core" of impervious earth or masonry placed near the centre of the dam. If impervious foundation is reached only at a considerable depth, this portion only of the embankment is carried to the extreme depth.

Various kinds of material can be used to make an embankment. Loam, sand, gravel, and clay, mixed in various proportions, are common. For the first three to be impervious they must contain a certain proportion of clay, the amount required depending upon the variation in size of the coarser particles. The suitability of a material for embankment construction can to some extent be determined by experiments. It should be strongly cohesive and plastic when mixed with water, and should be impervious; but the correct valuation of natural mixtures requires much experience in their actual use in construction.

If good material does not exist already mixed, artificial mixtures of gravel, sand, and clay may be used. A fairly uniform sand or gravel contains about 40 per cent of porous space. If then a mixture be made of coarse gravel, fine gravel, and sand, in each case just

enough of the finer material being used to fill the interstices of the next coarser, there will be in the mixture a porous space equal to $.40 \times .40 \times .40 = 6.4$ per cent, which will represent the proportion of clay necessary to make the mixture impervious. In practice it will take considerably more to insure the filling of all the interstices, as much as 15 or 20 per cent, depending upon the nature of the gravel mixture. In any case the percentage of voids in an artificial mixture can be readily determined by tests with water.

45. Core Walls. For a puddle wall of clay the minimum thickness ordinarily used is 4 to 8 feet at the top and about one-third the depth of water at the bottom, with a uniform batter on both faces. The trench is also usually made with a batter, the width at the bottom being one-third to one-half that at the ground level, with a minimum of 4 or 5 feet.

Instead of a core of puddle, many engineers prefer a core of rubble masonry or of concrete, made as impervious as possible. The advantages of this over a core of puddle are its safety against attack by burrowing animals, safety against wash in case minute leaks occur, and the greater certainty with which a concrete wall can be made impervious, especially where it joins the foundation.

Masonry core walls are made of various widths. Sometimes in case of embankments made of good material, they are made only a foot or two thick, their purpose being mainly to prevent the passage of burrowing animals. Ordinarily, however, a core wall is made 2 to 4 feet thick at the top, with a batter of ½ to ¾ inch per foot on each side down to the trench and then with vertical faces below. The height of a core wall should be equal to that of the highest water level.

Figs. 13, 14 and 15 show cross sections of several forms of embankments. Fig. 13 is without core wall except in the trench, Fig. 14 has a core wall of puddle and Fig. 15 one of concrete.

46. Dimensions of Embankments. On the water side the slope is usually protected from wave action and should only be sufficient to prevent slips. With coarse material this need not be flatter than 2 horizontal to 1 vertical. With finer material it may need to be 2½ or 3 to 1, or in some cases even 4 to 1, since earth in a saturated condition has a comparatively small angle of repose. On the lower side a slope of 2 to 1 is to be recommended, although 1½

to 1 has frequently been used. If the material will stand at 1 to 1, as broken stone, for example, then a slope of 1½ to 1 would be suitable. On high embankments, bermes placed 30 to 40 feet apart vertically are a desirable feature.

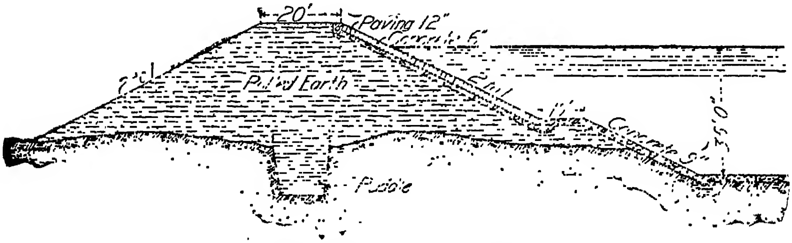


Fig. 13. Reservoir Embankment, Syracuse, N. Y.

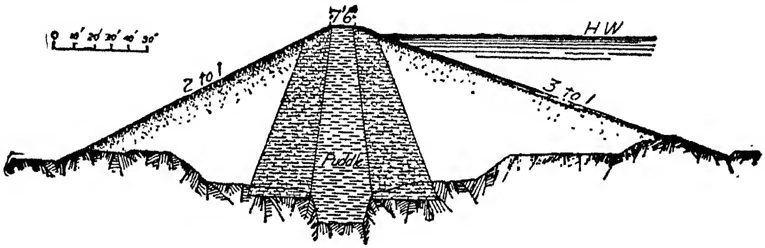


Fig. 14. Reservoir Embankment, Glasgow Waterworks.

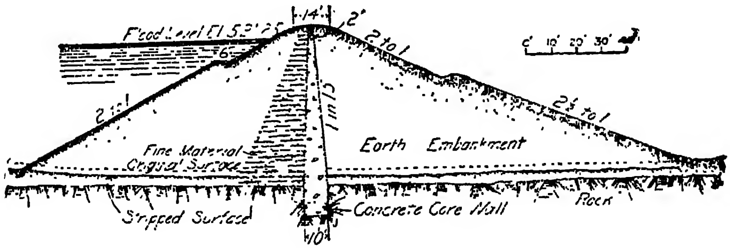


Fig. 15. Reservoir Embankment, Boston Waterworks.

The top of the dam should extend sufficiently above the high-water line to protect the material exposed to water action from frost and to give a safe margin against overflowing. This will be equal to the depth reached by frost plus an allowance of 2 to 5 feet for wave action, depending on the exposure to winds and the depth of the water.

The width of top is frequently fixed by the requirements for a roadway. Where not so fixed it is made to vary with the height—from 6 to 8 feet for very low embankments to 20 or 25 feet for embankments 80 to 100 feet high, or, approximately, width $= \frac{1}{5} h + 5$ feet, where $h =$ height of dam.

47. Construction. In preparing the foundation the surface soil must be removed over the entire site of the embankment to a depth sufficient to reach good sound material. All roots, stumps, and other perishable material must be removed, as any such material by decaying offers a passage for water. For the portion to be occupied by the core wall, if one is used, and a certain width in any case, the foundation must be excavated to an impervious stratum of solid rock or clay, and penetrate for a short distance such stratum. A sound bottom having been reached the surface should be roughened in order to give a better bond with the earth filling; and if the material is solid rock, all holes and crevices must be thoroughly cleaned and filled with cement or concrete.

After the foundation has been prepared the trench is first filled with the material selected. If puddle, it should be placed in 4- to 6-inch layers well rammed, or cut and cross cut with thin spades reaching well into the layer below, just enough water being used to render the material plastic. Where puddle is used in a narrow wall it is advisable to prepare it before placing by thoroughly pulverizing and tempering it with water, no more water being used than absolutely necessary. Puddle should be thoroughly worked and homogeneous. If concrete is used, special care must be taken to secure thoroughly good work in mixing and ramming, and in filling all irregular spaces in the excavation.

After the core is built to the surface, or a little above in the case of concrete, the main embankment is started. If the material used varies in quality, the finer and better should be placed above and adjoining the core wall, and the coarser placed on the down stream side and near the faces. If no core wall is used, the better material should still be placed in the up stream portion of the embankment. Stones exceeding 3 or 4 inches in diameter should not be allowed in the embankment except along the faces. The embankment is compacted usually by placing the material in layers 6 to 12 inches thick, wetting, and rolling with a heavy grooved roller weighing 200 to 300 pounds per lineal inch.

Much importance is attached to the work of compacting, and only by the best of supervision can work be secured. The use of water should be just sufficient to render the material plastic and capable of being packed, and no more. An excess of water makes rolling more difficult and increases subsequent settlement.

The up-stream slope must be protected from wave and ice action. This protection is usually afforded by a closely laid pavement about 18 inches thick laid on 6 to 12 inches of broken stone or gravel. Below low-water line a good layer of riprap is frequently substituted, the paving ending at a berme. The foot of the paving should be well supported by large blocks of stone or concrete. The down-stream face is usually sodded for sake of appearance and as a protection from rain, but may be protected by gravel and coarse material if more convenient.

48. Outlet Pipes. The design and construction of the outlet arrangements is one of the most important and at the same time most difficult features of the work. This is chiefly because of the difficulty of laying pipes or building masonry conduits through earth embankments in such a manner as to secure a perfect and reliable connection between the two materials. Poor work at this point is one of the chief causes of the many failures of earth embankments.

The outlet pipes are usually of cast iron and may either be laid underneath the embankment and surrounded thereby, or a culvert of masonry may be constructed in the embankment and the pipes laid therein, or they may be laid in a tunnel constructed in the natural ground at the end of the embankment or at some more remote point in the reservoir. A gate chamber containing the necessary valves is located at some point along the outlet pipe or conduit.

In the case of reservoirs with comparatively low embankments the outlet pipes are usually laid beneath the embankment at or near the lowest point. They should be laid on a good firm foundation in the natural ground, and should preferably rest upon and be surrounded by a bed of 8 to 12 inches of rich concrete, well rammed into the trench and left rough on the outside. To enable the earth to be more thoroughly bonded with the concrete, cut-off walls should be built projecting out from the main body of the concrete, 1½ to 2 feet, as shown in Fig. 16.

For some reasons an open culvert is much to be preferred to a simple pipe. Once constructed, additional pipes may be laid therein at any time; the pipes may also be readily inspected, and any leaks that occur in them do not endanger the structure, a matter of especial importance where the pipes are under heavy pressure. The same precautions must be taken in the construction of culverts as in the laying of pipes. They must have a good firm foundation and a good bond with the surrounding embankment. Imperviousness is secured by the use of a rich mortar and by plastering on the outside with Portland cement mortar neat or 1 to 1. Cut-off walls or projecting courses should be built around the outside at intervals as described for pipe outlets. At the connection with the gate house a cut-off wall is put in through which the pipes pass, and which must sustain the full head of water.

Figs. 16 and 17 show the two general methods of laying pipes through embankments.

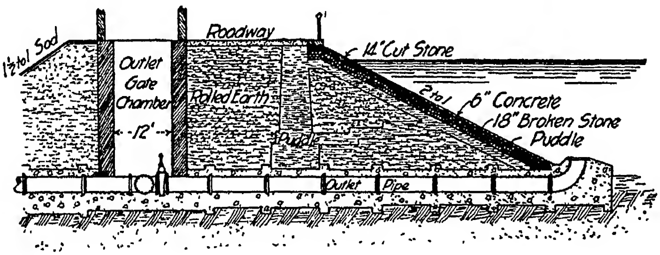


Fig. 16. Section Through Minneapolis Reservoir.

49. Gate Chambers. The gates or valves controlling the flow through the outlet pipes are placed in small masonry chambers, which, besides allowing of convenient operation of and access to the valves, also usually contain screening chambers and valve arrangements whereby water may be drawn from different levels. Gate chambers are preferably placed at or near the upper end of the outlet pipes in order that the pressure therein may be under control. They are, however, sometimes placed at the outer toe of the embankment, but this is undesirable, as it is impossible to shut off the water from the pipes in case of leakage except by the use of divers. Fig. 16 shows the gate chamber placed near the middle of the embankment.

while Fig. 17 shows it placed at the upper end of the outlet pipe. One advantage of the latter arrangement is that water may be drawn from different levels so as always to get water of the best quality. Fig. 18 shows a gate chamber for a small works located as in Fig. 17.

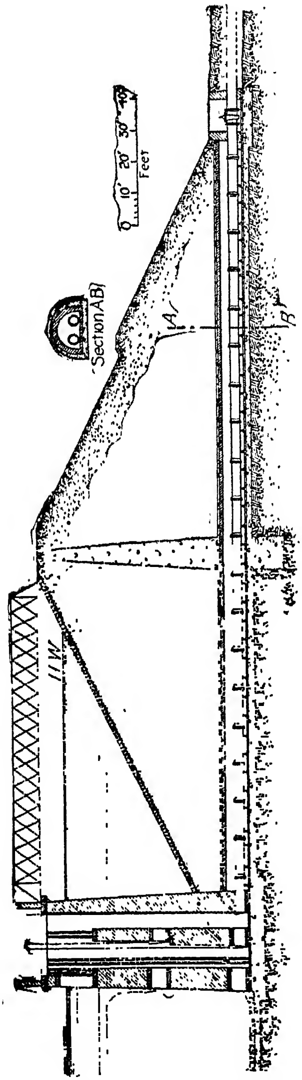


Fig. 17. Section Through Embankment and Gate Chamber.

The masonry of the chamber is usually of heavy rubble, faced with ashlar and lined with hard brick or cut stone. It should be laid in rich Portland-cement mortar to secure imperviousness. The walls will vary in thickness with their unsupported length, or the size of interior chamber, but the exterior walls are usually made 3 to 4 feet thick at the top, with an increase of about three-fourths inch to 1 inch in thickness per foot of depth, the batter being made on the outside for convenience and to furnish a better bond with the earthwork. Interior walls may be made of slightly less thickness. The foundation should be prepared with great care, as unequal settlement is liable to occur, causing cracks in the masonry of the culvert and displacing the outlet pipes.

Fish screens are usually copper-wire screens with $\frac{1}{8}$ to $\frac{1}{4}$-inch mesh, fastened to wooden or iron frames and arranged to slide in grooves in the masonry. They are arranged in pairs, and each screen is made up of several sections of a size convenient to handle.

The gate chamber is surmounted by a gate house in which is located the operating mechanism of valves and screens. As this building is frequently quite prominent,

it is important that it be given an artistic treatment suited to the surroundings.

50. Waste Weirs. As already noted, one of the most fruitful causes of reservoir failures is insufficiency of waste weir capacity, resulting in the overflowing of the dam and its rapid destruction. Mention need only be made of the terrible Johnstown disaster in 1889, where, on account of insufficient wasteway, an earthen embankment was destroyed, resulting in the loss of over 2,000 lives and the destruction of property valued at 3 to 4 million dollars.

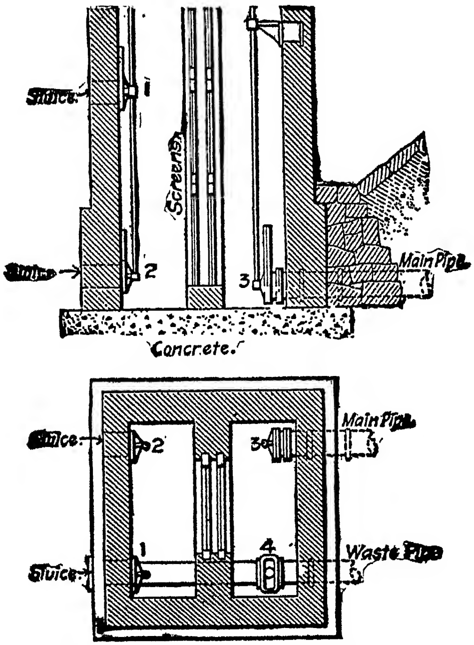


Fig. 18. Gate Chamber, Ipswich, Mass.

In section 14 the subject of maximum flood flows was fully discussed. The maximum flood having been estimated, it remains to provide some safe means whereby it may be passed to the valley below.

This is done in three different ways: (1) A wasteway may be excavated in the natural ground at one or both ends of the dam. Where the foundation is of rock this is a very safe and effective form of wasteway.

(2) The wasteway may sometimes be formed at some low point in the dividing ridge, and the water led to another valley.

(3) The third form of wasteway is provided by making a portion of the dam of masonry designed as a spillway, and placed at about the axis of the valley. The forms of such dams are discussed in detail in section 54. At the junction of the masonry and the earth portions, the lower slopes of the embankments must be retained by heavy wing walls built out from the masonry dam.

The requisite capacity being known, the length and depth of weir are to be determined. Either may be assumed and the other computed by means of the weir formulas as given in "Hydraulics." Weir heights will ordinarily range from 2 to 4 or 5 feet, with lengths of 50, 100, or even 500 feet, or more, depending on the required capacity. In any case the flood line determines the height of the main part of the dam, while the weir crest determines the storage capacity.

MASONRY DAMS.

51. General Conditions. Dams of masonry can safely be built only upon very firm foundations. Low dams of a height of 20 or 30 feet, and occasionally higher, have been founded on firm earth, but high masonry dams should be constructed on nothing less substantial than solid rock. In any case it is necessary to prevent practically all settlement, for with a material such as masonry any appreciable settlement is quite certain to cause cracks.

Masonry dams are designed in accordance with theoretical considerations so as to fulfill the following conditions: (1) The dam must not overturn or slide on its foundation, and (2) the pressures in the dam or foundation must be within safe limits.

The first consideration will govern the design of all dams up to a height of 100 feet or more and are therefore the only considerations which will be taken into account here.

Dams up to 30 or 40 feet in height are usually made trapezoidal in form, the saving obtained by making the faces curved or on broken lines not being enough to justify the extra trouble.

Let A B D C, Fig. 19, be a section of a trapezoidal dam. Let the dimensions be as represented in the figure. Further, let $w =$ weight of a unit volume of water, and $w' =$ weight of a unit volume

of masonry. Let g = specific gravity of the masonry $= \frac{w'}{w}$.

The water pressure is represented by P and the weight of the dam by G, and it is assumed that the water level is at the top of the dam We will consider a length of dam of one foot. The height is known and the top width a and front batter m are assumed. Usually the front face A C is made vertical, or at a slight batter of one inch to the foot or so. In the former case $m = 0$ and in the latter $m = \frac{1}{12} h$.

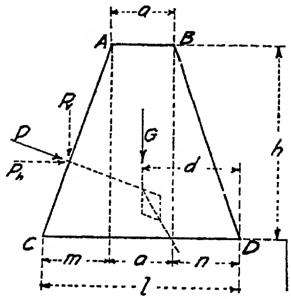


Fig. 19. Trapezoidal Dam.

From principles of mechanics we find the following value of the width of base l,

$$l = \sqrt{A + B^2} - B \qquad (3)$$

in which

$$A = a^2 + 2\,a\,m + \frac{h^2}{g} + \frac{m^2}{g}$$

and

$$B = \frac{m}{g} - \frac{m}{2} + \frac{a}{2}$$

In solving problems the numerical values of A and B should first be obtained and these values then substituted in the formula (3) for l.

If $m = 0$, as for a vertical face, then

$$l = \sqrt{\frac{5}{4}\,a^2 + \frac{h^2}{g}} - \frac{a}{2} \qquad (4)$$

Example. What width of base will be required if $h = 20$ ft.; $a = 5$ ft.; and the weight of masonry be considered 2.3 times the weight of water, or $g = 2.3$. Let value of m be 2 feet, giving a batter of 1 in 10.

Getting first the values of A and B we have $A = 5 \times 5 + 2 \times 5 \times 2 + \frac{20 \times 20}{2.3} + \frac{2 \times 2}{2.3} = 221$. $B = \frac{2}{2.3} - \frac{2}{2} + \frac{5}{2} = 2.37$.

Then $l = \sqrt{221 + 2.37 \times 2.37} - 2.37 = 12.7$ ft. Ans.

For dams exceeding 30 or 40 feet in height, it is economy to build the lower face in the form of a curve or broken line. The simplest way of calculating the section of such a dam (up to a height of 100 feet at least) is to treat it at first as similar to the form previously considered, but with a vertical upper face and top width of 0. Then the formula for bottom width becomes

$$l = \frac{h}{\sqrt{g}} \qquad (5)$$

This gives the triangular section A B C in Fig. 20. This form can then be modified by adding a suitable width a at the top and joining the point F with the sloping face A C by a smooth curve F D.

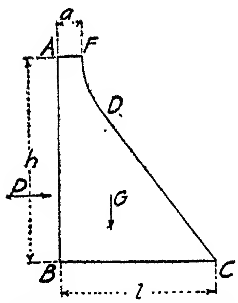


Fig. 20. Curved-Face Dam.

52. Top Width and Height Above Water Line. If the dam is to be used as a driveway, the top width will have to be at least 8 feet besides width of parapets. Otherwise the width and height above high-water line must be such as to secure stability against wave and ice action as just noted, and to prevent waves from washing over the top. In practice the width varies from a minimum of 4 to 5 feet for low dams to 15 or 20 feet for very high dams; and the height above high-water line from 2 or 3 feet to about 10 feet. In some cases much larger dimensions may be required for low dams than those given.

53. Construction. For large dams the foundation should be solid rock. In preparing the foundation surface all loose and partially decomposed material should be excavated until a firm base is reached. If the bottom is smooth it should be roughened by excavating shallow cavities in the rock. At points where crevices occur the excavation must be carried down to a solid bottom and all loose material must be removed. After an acceptable surface is reached it should be thoroughly washed or scrubbed with water in order that there may be a secure bond between the foundation and the masonry.

Uncoursed rubble or concrete is usually employed in dam construction. The object to be attained is to secure a homogeneous structure, free from all through joints or weak places of separation.

Concrete, well placed, is in this respect an ideal material. Rubble masonry, in which all joints are thoroughly filled with mortar, and larger spaces with concrete, has been used for most of the high dams.

In constructing the masonry the principal points to be emphasized are clean surfaces, irregular surfaces, joints absolutely filled with compact mortar, great care to give good bedding, and constant supervision. Mortar and cement should be thoroughly rammed into all spaces, using for this purpose suitable forms of rammers.

In the construction of dams of moderate height, earth backing is often carried up to the water level with a slope of 2 or 3 to 1, as in an earthen dam. If a dam is located on a porous or bad foundation or on one of earth, a good, compact backing will much reduce the percolation under the dam, and therefore the tendency of any upward pressure, and will add considerably to the safety of the structure. It is especially applicable to spillways in earthen embankments.

The arrangements for drawing water from the reservoir are similar in general to those described in the last chapter. The outlet pipes are built in the masonry at or near the lowest point of the dam, and terminate in a gate chamber constructed just above and in connection with the dam. The gate chamber has the same functions as explained in the case of earthen embankments. No danger is here to be apprehended from constructing the pipes in the body of the dam.

54. Masonry Waste Weirs. Masonry dams are not usually designed to allow water to pass over their entire length, but a certain portion only is made to act as a waste weir. The form of a masonry weir depends much upon local conditions, chief of which are height of dam, character of foundation, amount of ice and driftwood to be expected, and quantity of water to be provided for. A weir is essentially a dam with its top and lower face so constructed as to permit the water to pass over it without damage. Besides the design of the profile, the protection of the stream bed below the dam is a very important feature, as many dams have been undermined by failure at this point even where the bed has been solid rock.

Figs. 21, 22 and 23 show three forms of waste weirs. Fig. 21 permits the water to fall vertically and is suitable for small heights; Fig. 22 is preferable for larger quantities of water and greater heights; while Fig. 23 represents a form of construction suitable for the largest dams.

55. Timber Dams. Where a dam is constantly submerged, a timber structure is of a permanent nature, and will need repairs only on account of the wear of the apron. A timber dam may also be advisable in certain circumstances even when its life will be short, as, for example, where a temporary supply may be furnished pending the construction of more permanent works, or where the expense of permanent and costly structures is for the present prohibitory.

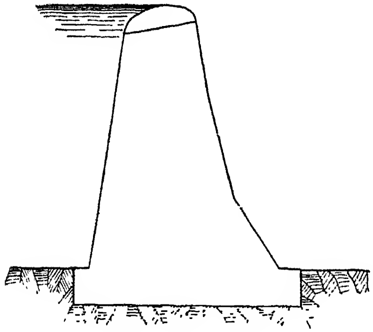


Fig. 21. Waste Weir.

Such dams are, however, used mostly for diversion purposes or for water power, and seldom for the storage of large volumes of water.

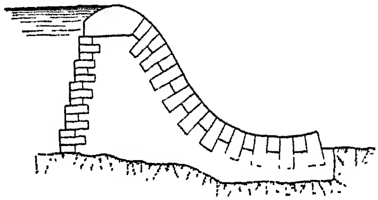


Fig. 22. Waste Weir.

Timber dams may be constructed on any kind of a foundation, but are usually built on rock or on a gravelly bed. They consist of cribs or frames built of logs or squared timber, filled with stone and clay, and planked over to render them water-tight. They may be built as separate cribs in sections, each section consisting of perhaps 3 to 4 cribs, or as one continuous framework. The former method is especially useful in dealing with large flows and irregular foundations, the stream being gradually closed as the sections are constructed. The cribs may also be filled and sunk separately so as to form piers on which a continuous structure may be built.

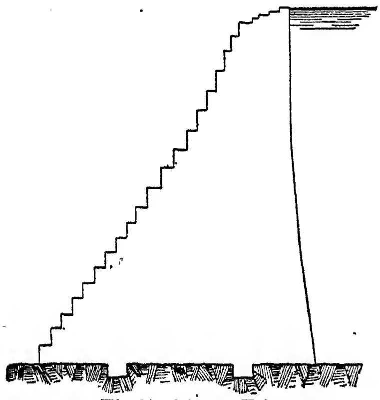


Fig. 23. Waste Weir.

The foundation of a crib dam, if soft, is prepared by dumping stone over the area to be built upon. In the framed dam the founda-

tion must be more carefully prepared. Where it is soft the dam is supported on piling, and sheet-piling is used to prevent underflow. If the dam is built on a rock bottom, it must be bolted thereto. The framework is usually built with a sloping upper face and a series of

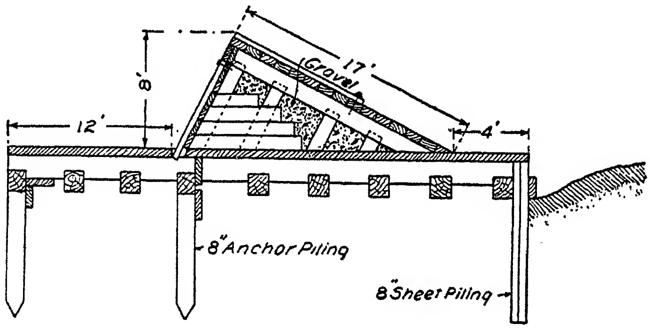


Fig. 24. Timber Weir.

stepped aprons below, or a single free wall to a water cushion. Rock and gravel, or puddle, is used for filling. Fig. 24 shows a form of dam on pile foundations and Fig. 25 a dam anchored to solid rock.

56. Loose Rock Dams. Dams composed largely of loose rock have been used to a considerable extent, and in some respects present

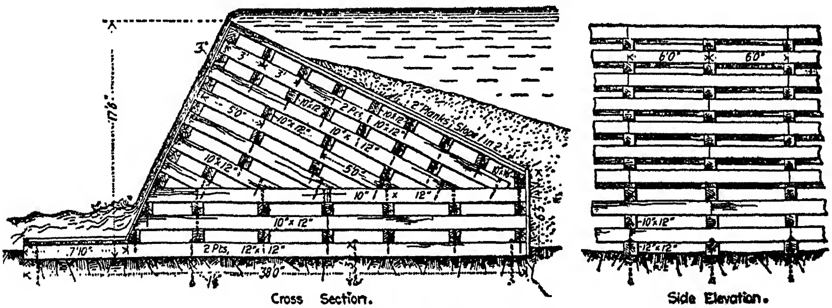
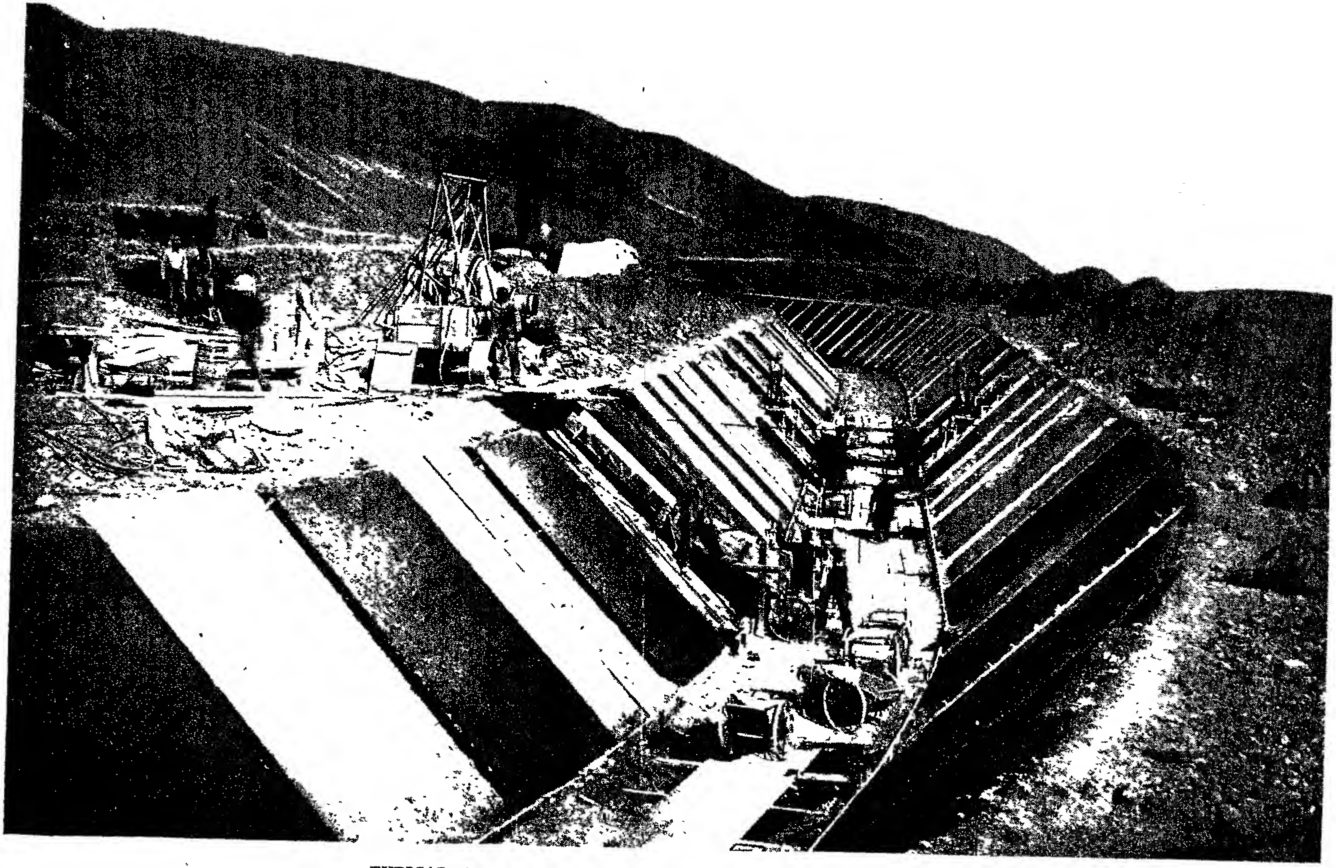


Fig. 25. Dam Anchored to Rock.

considereble advantages as to stability. Another advantage is that they can be constructed in running water, but the finished dam is not suited to act as a waste weir.

The body of the dam is made of loose rock placed with more or less care, and rendered comparatively impervious by a sheathing of

plank, or by a facing of earth or fine material on the upper face, or, as in one case, by a core of steel. As regards stability the principle of construction is of the best. Since considerable percolation is likely to take place, such a dam cannot be founded on a material liable to scour; and if the dam is high, the foundation should be solid rock. The lower slope is usually 1 to 1, while the upper slope may be made $\frac{1}{3}$ or $\frac{1}{2}$ to 1; but to secure these steep slopes it is necessary to lay the stone for a considerable thickness as a dry wall. Above this wall the facing of timber or earth is placed. The former material is objectionable on account of its perishable nature.



TYPICAL VIEW OF LOS ANGELES OWENS RIVER AQUEDUCT
The length is 217 miles. The view shows method of constructing concrete panels.
Courtesy of Municipal Engineering and Contracting Company, Chicago, Illinois

WATER SUPPLY.

PART II.

CONDUITS AND PIPE LINES.

PIPES.

57. **Materials and Stresses.** Where the source of supply is at a considerable distance from the place of consumption the design and construction of the necessary works for conducting the water is a matter of great importance and demands special consideration. Usually a distant source is at a higher elevation than the city to be served, so that it will be possible to convey the water partly or wholly by gravity. In many cases, however, a part or the whole of the water will require pumping, so that the design will also involve a study of possible pumping arrangements. It will usually be necessary to consider several designs based upon different locations and often upon different types of conduits.

A variety of materials may be employed for the construction of water conduits. If the conduit is not under pressure, the form of construction used may be an open canal dug in the natural earth, or a masonry conduit in "cut and cover," or a tunnel. Where the water flows under pressure the first two types are not suitable and a pipe, or possibly a tunnel, must be employed.

The materials used for water pipes are cast iron, wrought iron, steel, wood, cement, vitrified clay, lead, and occasionally a few other materials. The important requirements for a water pipe are strength, durability, and low cost. The relative importance of these requirements will vary under different circumstances, and this will lead to the use of different materials in different cases.

The tensile stress in a water pipe under pressure is given by the formula in section 10, of Hydraulics,

$$S = \frac{pr}{t} \qquad (6)$$

where S = tensile stress per sq. inch
p = pressure in lb. per sq. in.
r = radius of the pipe in inches,
t = thickness of pipe in inches.

If f represents the safe tensile strength of a pipe material, then the required thickness to resist the pressure will be given by the formula

$$t = \frac{pr}{f} \tag{7}$$

Example. What will be the required thickness of a steel pipe 3 ft. in diam. for a water pressure of 100 lb. per sq. in. assuming the safe stress = 10,000 lb. per sq. in.

From formula 7, $t = \dfrac{100 \times 18}{10,000} = .18$ inch. Ans.

Besides the stress due to steady water pressure, the pipe must be strong enough to resist the shocks due to the sudden stoppage of flowing water, called *water hammer*, also the pressure of the surrounding earth and the action of other outside forces, changes of temperature, and blows and shocks received in transportation and construction. The stresses due to these additional forces cannot be accurately calculated; but they are allowed for in practice by various empirical rules.

58. Cast-Iron Pipe. Cast iron is the most widely used material for water pipe. By reason of its moderate cost, its durability, and the convenience with which it may be cast in any desired form it is almost universally employed for the pipes and various special forms of distributing systems. It is also frequently employed for large pipe lines, and is now easily obtained in any desired diameter up to 6 feet or more. Cast-iron pipes are made in lengths of about 12 feet, which are joined together usually by the bell-and-spigot joint run with lead. Branches and other irregular forms are used for connections. These are called special castings, or simply "specials."

A formula for the thickness of cast-iron pipe applicable to diameters up to 3 feet is as follows:

$$t = \frac{(p + 140 - 4r)}{3300} r + 0.25 \tag{8}$$

where t = thickness in inches;

p = static pressure in pounds per square inch;

r = radius of pipe in inches;

0.25 = allowance for eccentricity, deterioration, and safety in handling.

In Table No. 10 are given the thicknesses of pipe for various pressures, also the average weight per foot, and the total weight of 12-foot lengths, as employed by the Metropolitan Waterworks, of Boston.

TABLE 10.

Thickness and Weight of Water Pipe.

Diameter in Inches.	Class A. 115-foot Head.			Class B. 150-foot Head.			Class C. 200-ft. Head.			Class D. 250-ft. Head.			Class E. 300-ft. Head.		
	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.
4	....	......	......	....	......	....	....	....	....	0.40	19.2	230	0.45	21.2	255
6	....	......	......	....	......	....	....	....	....	0.46	31.7	380	0 50	34.6	416
8	....	......	......	....	......	....	....	....	....	0.52	47.1	565	0.55	50.0	600
10	....	......	......	....	......	....	....	....	....	0 60	66.7	800	0 63	70.0	840
12	....	......	......	0 57	75 8	910	0 61	80.8	970	0.65	85.8	1030	0.69	91.2	1095
14	....	......	......	0.61	94.2	1130	0.65	100.0	1200	0 70	107.5	1290	0.75	115 0	1380
16	....	......	......	0.65	115 0	1380	0 70	123.7	1485	0.75	132.5	1590	0.81	142 9	1715
20	....	......	......	0.73	160.8	1930	0.79	173.3	2080	0.85	186 2	2235	0.92	201 2	2415
24	....	......	......	0.80	210 4	2525	0.87	228 3	2740	0.95	248.7	2985	1.03	269.2	3230
30	....	......	......	0.92	302 1	3625	1.00	327.5	3930	1 10	361.2	4335	1 20	393.3	4270
36	0.93	366 7	4400	1.03	404.2	4850	1.13	441.7	5300	1.25	491.7	5900	1.36	533.3	6400
42	1.01	467 5	5610	1 14	524.2	6290	1.27	581.2	6975	1.40	645.8	7750	....	......	......
48	1.15	605.8	7270	1.25	655.8	7870	1.40	730.0	8760	1.55	818.3	9820	....	......	......
54	1.23	730 8	8770	1.35	797 5	9570	1 53	919.2	11030	....	......	......	...	......	......
60	1.35	885 8	10630	1.50	979.2	11750	1.70	1132.5	13590	....	......	......	....	......	......

59. Joints. The joint which is ordinarily employed in this country is the bell-and-spigot joint. The space between bell and spigot is filled with lead, which is calked solidly into place so as to be water-tight. Many forms of bell or socket have been devised, but practice has come to be quite uniform on this point.

In Table No. 11 are given various dimensions of standard bell and spigot of the Metropolitan Waterworks (Fig. 26), together with amounts of lead and packing required per joint.

The ordinary bell-and-spigot joint with lead packing will enable pipes to expand and contract under moderate changes of temperature such as occur with buried pipes.

Curves of large radius can be constructed with straight pipe by deflecting each length slightly. In this way it is possible, with a reasonable deflection, to lay 4 to 8-inch pipe to a curve of 150-foot

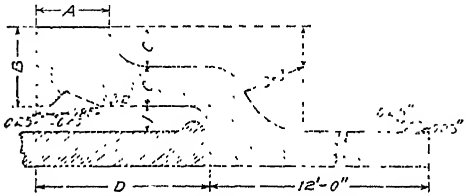


Fig. 26. Standard Bell and Spigot.

radius, a 16-inch pipe to a 250-foot radius, and a 36-inch pipe to a 500-foot radius.

TABLE 11.

Dimensions of Standard Bell and Spigot, Metropolitan Waterworks, Boston (Fig. 26).

Size of pipe.	Class.	Dimensions in inches.					Average wt. of lead per joint.		Weight of jute gasket per joint.
		A	B	C	D	J	With gasket.	Solid lead.	
4	All classes	1.50	1.30	0.65	3.00	0.40	7	9¼	.10
6	"	1.50	1.40	0.70	3.00	0.40	9¾	12¾	.15
8	"	1.50	1.50	0.75	3.50	0.40	12½	18¾	.25
10	"	1.50	1.50	0.75	3.50	0.40	15¼	23¼	.30
12	"	1.50	1.60	0.80	3.50	0.40	18	27	.35
14	"	1.50	1.70	0.85	3.50	0.40	20½	31	.40
16	"	1.75	1.80	0.90	4.00	0.50	31¼	50½	.65
20	"	1.75	2.00	1.00	4.00	0.50	38½	62	.80
24	"	2.00	2.10	1.05	4.00	0.50	45½	74	.95
30	B and C	2.00	2.30	1.15	4.50	0.50	56	100½	1.55
30	D and E	2.00	2.50	1.25	4.50	0.50	57	102	1.55
36	A, B, and C	2.00	2.50	1.25	4.50	0.50	67	120½	1.85
36	D and E	2.00	2.80	1.40	4.50	0.50	68½	122½	1.85
42	A, B, and C	2.00	2.80	1.40	5.00	0.50	77½	154	2.60
42	D	2.00	3.20	1.60	5.00	0.50	78½	156	2.60
48	A, B, and C	2.00	3.00	1.50	5.00	0.50	88½	176	3.00
48	D	2.25	3.50	1.75	5.00	0.50	89½	178	3.00
54	A and B	2.25	3.10	1.55	5.50	0.50	99½	215	3.95
54	C	2.25	3.90	1.95	5.50	0.50	100	215½	3.95
60	A and B	2.25	3.20	1.60	5.50	0.50	110½	239	4.40
60	C	2.25	4.20	2.10	5.50	0.50	111	241	4.40

60. Special Castings. The ordinary special castings required are the ¼, ⅛, and $\frac{1}{16}$ bends or curves, T's and crosses, or three-way and four-way branches, Y branches, blow-off branches, offsets,

sleeves, caps, and plugs. The various forms are illustrated in Fig. 27. Many of the larger cities have their own standard designs for specials as well as for straight pipe, which differ more or less from the manu-

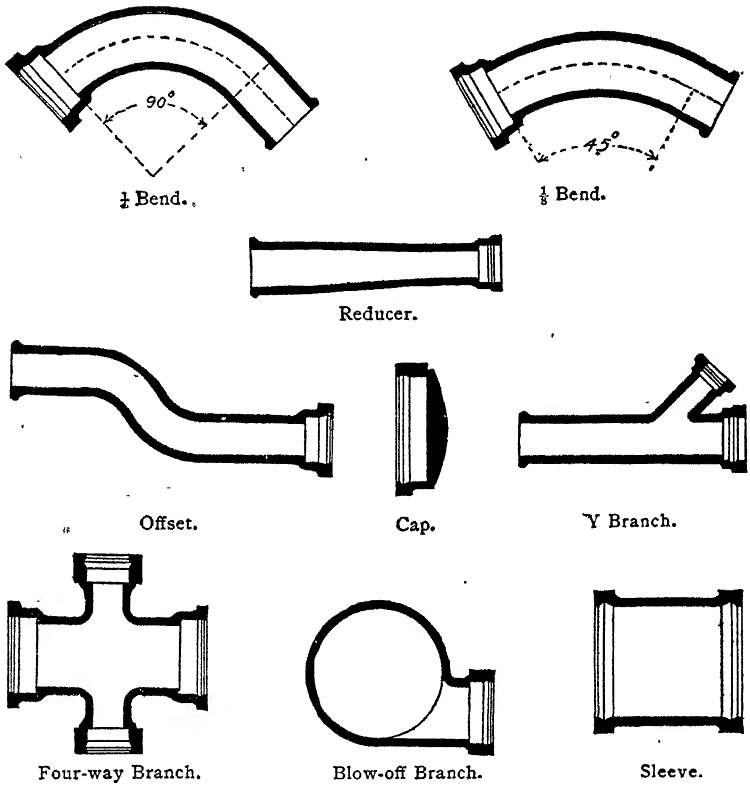


Fig. 27. Special Cast-iron Pipe Fittings.

facturers' standards. For the smaller cities it will be much the more economical to use either the manufacturers' standards or those of some neighboring large city.

The various branches are manufactured either with part bell and part spigot ends, or with all bell ends. The latter form is usually preferred for branches, as it enables connections to be readily made by means of pieces of pipe.

61. Wrought Iron and Steel Pipe. Wrought iron and steel have been used to a considerable extent for water pipes, and for large

pipe lines these materials present considerable advantage over cast iron. Since steel is much stronger than cast iron, the use of it will give a much lighter pipe, an advantage as regards transportation, but a disadvantage as regards durability, especially for small sizes. Special forms are not so readily constructed of steel, so that for distributing mains cast iron is much preferable. Another disadvantage of steel pipe is that with the ordinary riveted joints a considerably larger pipe is required than if a smooth cast-iron pipe is used on account of the increased friction.

Small sizes of pipe may be made by means of the lap-welded joint, or the spirally-riveted joint, or the longitudinal lap-riveted joint. Such pipes are made in sections of 12 or 15 feet which are connected in the field in various ways, such as by a screw coupling, or by means of a cast-iron bell and a spigot consisting of a steel or wrought-iron band, or by riveting, or by merely driving the sections together. For large sizes riveted longitudinal and circular joints are usually employed. Single sheets are bent and riveted to form one section of pipe, which may be made either cylindrical in form, or made with a slight taper and the sections put together stove-pipe fashion. The design of the riveting is too large a subject to be taken up here.

Changes in direction are usually made by forming one or more joints at a small bevel. Two or three standard bevels of small angle may be adopted, and any desired curve made by the use of one or more of these bevels. Branches, etc., for the ordinary sizes of pipes, are usually made of cast iron and are riveted or bolted firmly to the steel pipe. Valves are joined to the pipe in a similar manner by means of cast-iron flanges.

62. Wooden Pipe. The manufacture of bored pipe for water mains has been somewhat revived in recent years, and a considerable amount of such pipe is now manufactured under the name of "improved Wyckoff pipe." The pipe is made from solid logs, but it depends for strength upon spiral bands of flat iron which are wound tightly about it from end to end. The exterior of the pipe is coated with pitch as a protection to the bands. The joints are made by means of wooden thimbles fitting tightly in mortises in the ends of the pipe, and, in laying, the sections are driven together by means of a wooden ram. The interior surface is smooth and continuous

The pipe is made in sections 8 feet long, and in sizes from 2 to 17 inches in diameter. The bands are spaced according to the pressure. Branch connections are made by means of cast-iron specials which have long sockets into which the wooden pipe is driven. About 1,500 miles of this pipe is reported to be now in use. It is very durable and is said to cost somewhat less than cast iron where the transportation charges are not excessive. Wooden stave pipe is another form that has been extensively used in the West.

The durability of wooden pipe is chiefly a question of the life of the bands. Wood, itself, when kept saturated with water, has an almost indefinite life, old water mains in Philadelphia, New York, and Boston having been found perfectly sound after sixty or seventy years of use.

63. Vitrified Clay Pipe has been employed in a few places for conduits. It is cheap, indestructible, and when the joints are carefully made the leakage is very small. It is generally used under no pressure, but in one or two instances has been designed to carry considerable pressures.

64. Materials for Service Pipes. Service pipes, or pipes for conducting water to individual consumers, are made of a considerable variety of materials. Galvanized, tin-lined, lead-lined, and cement-lined iron pipe are widely used, but the most common is lead pipe. Lead pipe is practically indestructible, but rather expensive and heavy for high pressures. In some places it cannot be used with safety on account of the danger of lead poisoning. Certain waters only will attack lead to a sufficient extent to render its use dangerous, but, despite the study that has been put upon the subject, it is not yet fully known, without actual experiment, what effect various classes of waters will have.

Tin-lined pipe is now being used to a considerable extent. It is quite expensive, but the experience with it so far indicates that it may be very durable.

CONSTRUCTION OF CONDUITS.

65. Classes of Conduits. Conduits are divided into two general classes: (1) those in which the water surface is free and the conduit therefore not under pressure, and (2) those flowing under pressure. To the first class belong open canals, flumes, aqueducts,

and usually tunnels, and to the latter belong pipe lines of iron, steel, wood, or other material capable of resisting hydraulic pressure, and sometimes tunnels. Conduits of the first class must obviously be constructed with a slope equal to that designed for the water surface, or equal to the hydraulic gradient. This will be a very light and uniform slope, and such conduits will therefore often require in their construction long detours to avoid hills and valleys, or resort must be had to high bridges, embankments, cuttings, or tunnels. Conduits of the second class may be constructed at any elevation below the hydraulic grade line.

Long conduits usually include both masonry aqueducts and pipe lines, each class being used where most suitable. The former is used as a rule where the ground lies near or above the hydraulic grade line, and the latter where it lies below for any considerable distance. High and long aqueduct bridges are no longer built; a pressure conduit being substituted, which may follow the ground profile closely.

66. Canals. The open canal is not often used for conveying water for city use, but for irrigation purposes it is the common form of conduit. For the former purpose it has several objections, such as loss of water by percolation and evaporation, exposure of water to pollution from surface drainage and otherwise, and exposure to summer heat, which not only warms the water but promotes vegetable growth. However, where a canal can be constructed with little cutting or embankment, and where the material is nearly impervious, it may be the best form of construction.

The allowable velocities for unprotected canals vary from about $1\frac{1}{2}$ to 2 feet average velocity for light sandy soils, $2\frac{1}{2}$ to 3 feet for ordinary firm soils, and 3 to 4 feet for hard clay and gravel. In rock or hardpan 5 to 6 feet may be allowed. A velocity of 2 to 3 feet per second is sufficient to prevent silt deposits and the growth of weeds.

The velocity and discharge for any given slope and cross-section is calculated from Kutter's formula as explained in Hydraulics. In using this formula the selection of a proper value of n is a matter of much uncertainty. For unlined channels it is usually taken at .020 to .025. If vegetation is allowed to accumulate in the canal, a large allowance must be made for increased resistance caused thereby. The cross-section of a canal is usually trapezoidal in form. Fig. 28

shows a section built principally by embankment. Clay puddle is placed in the center of each embankment.

Side slopes in ordinary soils will vary from 1 to 1 for hard clay and gravel, to 3 to 1 or 4 to 1 for fine sand. The tops of the bank should be from 1 to 2 feet above the water line. If the soil is very porous, a lining of concrete or puddle may be necessary.

At sharp bends, and wherever the velocity exceeds the safe velocity for the material, some form of revetment is necessary. This may be merely a layer of gravel, or a paving laid dry or in cement, or a layer of concrete, according to the velocity of the water.

Waste weirs and sluice gates should be provided at intervals along the canal to prevent flooding and to permit of rapid emptying, but the flow in the canal is regulated for the most part by sluice gates at the head of the canal. These and other forms of canal gates are supported either by masonry walls or by timber framework.

Canals are carried across valleys on trestles or bridges, or, in the case of short crossings, on embankments with a culvert or arched

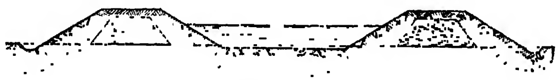


Fig. 28. Canal Section in Embankment.

bridge beneath. Under crossings are made by means of inverted siphons of pipe.

67. Masonry Conduits. For conveying relatively large quantities of water over territory where the conduit can readily follow the hydraulic grade line, the masonry conduit is a preferable form of construction. If properly constructed it is very durable, requires little attention, and if the topography is favorable it is much cheaper than large pipe conduits of iron or steel. Masonry is unsuited to withstand tensile stresses, hence it is not used to convey water under pressure. Masonry conduits should not often be employed for cross-sections less than 10 or 15 square feet, for, unless the location be very favorable, their cost for such small sizes is likely to be greater than that of steel or iron pipes. The velocity should preferably be such as to prevent deposit of sediment, which requires 2½ to 3 feet per second average rate; and for brick or concrete masonry it should not exceed 6 or 7 feet per second. Higher velocities may be allowed

if stone masonry of hard material is employed, or if a lining of iron or steel is used. If sufficient head is available, a smaller conduit will result if the velocity is made as large as the material will stand without danger of excessive wear.

Kutter's formula is usually employed in these calculations. (See Hydraulics.) The value of n to be used will vary with the character of the masonry about as given in Hydraulics.

Brick is the most suitable material for linings, and is commonly used also for the entire arch crown. For the side walls and foundation, rubble masonry or concrete is employed. For places of great wear paving brick in cement is a good substitute for granite. Concrete is better suited than either stone or brick for irregular forms and especially for light sections. For small aqueducts a rectangular form has often been used, as in Fig. 29, the cover being of stones, slabs, or arches. For large sizes the horseshoe shape is better adapted, as shown in Fig. 30, which represents the form adopted for a large conduit for Boston.

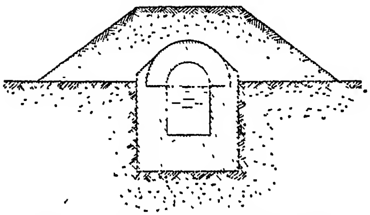


Fig. 29. Gallery, Vienna Waterworks.

The thickness of the arch is made about one-tenth to one-sixth the width of the opening, and of two, three, or four rings of brick, or a corresponding of concrete, depending on span and weight of covering. The arch is generally segmental in form. The invert, in compact ground, is made only thick enough to secure a firm and impervious bottom, two or three rings of brick, or a thin layer of concrete with brick lining, being usually employed. A timber foundation and sometimes piling may be required on soft soils. Settlement must be reduced to very low limits, or cracks and leakage will result. It is unnecessary to state that in work of this kind the masonry must be constructed with the most careful supervision. Concrete and stone masonry should be

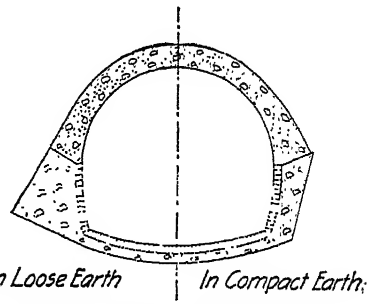


Fig. 30. Forms for Large Conduit.

given one or two finishing coats of thin neat cement to secure imperviousness, the last coat to be finished as smooth as practicable. If carefully done, and no settlement occurs, the leakage will be slight.

The aqueduct should be covered to a depth of 3 or 4 feet to prevent the formation of ice and to protect the masonry. Embankments should be given a slope of $1\frac{1}{2}$ to 2 horizontal to 1 vertical according to the nature of the material. They should be trimmed to a rounded outline and then sodded.

Culverts for crossing small streams, and bridges for larger ones, are a part of the design. Some of the most monumental works of history are the bridges which have been built for carrying aqueducts. Large aqueduct bridges are now seldom constructed, pipe lines being substituted, but bridges of moderate size will often be the more economical design.

68. Pipe Lines. As to location, a pipe line must follow in general the variations of the ground surface, and such a location should be selected as will enable it to do so and at the same time give low pressures.

Where the total available head is fixed, the size required for any given capacity is readily determined by the table of the flow of water in pipes in Hydraulics. In case the water contains suspended matter, it is desirable to maintain a self-cleansing velocity of 2 to $2\frac{1}{2}$ feet per second, otherwise the sediment must be blown out at frequent intervals. If the line is divided into sections by reservoirs or overflows, the size of each section is determined independently of the others.

If pumps are used to force the water through the pipe then the proper size depends on the relative cost of pipe, and of pumping against an increased head. A large pipe gives low friction head of the water and therefore saves in pumping expenses, but a large pipe is more expensive than a small one. In general it may be assumed that a proper size of pipe is one which calls for a velocity of flow of from $1\frac{1}{2}$ to $2\frac{1}{2}$ feet per second, the former value for pipes of 6 to 12 inches in diameter, and the latter for pipes of 3 or 4 feet in diameter.

69. Laying of Pipes. Trenches for water pipe are not usually deep enough to require much bracing or sheeting, the depth being ordinarily only sufficient to give the necessary covering. Deep trenches will, however, be required occasionally, as where the pipe line crosses a high ridge extending above the hydraulic gradient.

The laying of cast-iron pipe is usually begun at a valve or special. Small pipe up to 6 or 8 inches in diameter is easily handled without a derrick, the sections being lowered into the trench by two or three men. In laying, care should be taken to enter the pipe to its full depth and to see that there is sufficient joint space all around. If special strength is not required, this packing may nearly fill the space back of the enlargement or V-shaped space in the bell. The remaining space is filled with molten lead. In pouring the joint the lead is guided into the space by a jointer, commonly made of clay formed around a length of rope. This is placed about the pipe so as to press against the hub, except at the top, where an opening is made for pouring. Patent jointers are better for large pipe and difficult work. After pouring, the lead is loosened somewhat from the pipe by means of a chisel and set up by calking iron and hammer. To do good work there should be plenty of room around and under the pipe. In wet trenches and with small pipe, two or three sections may be joined before lowering. Riveted pipe should be connected up in as long sections as practicable before being transported to the trench, so that as much of the riveting may be done by power riveters as possible.

When placed in the trench the pipe should have an even bearing on firm soil or on blocking, and should be well supported while the joints are being riveted. The riveting is usually done by hand, but power riveters have been used in a few cases. After riveting, all field joints should be calked, and these and all other abraded places painted. Some re-calking may be needed after the pipe is tested.

In constructing the pipe system one of the most important points to settle is the depth at which the pipes should be laid. In warm climates a covering of 2 to 3 feet is sufficient. In cold climates the depth to be adopted is that which will be sufficient to prevent freezing. In a general way it may be stated that for a latitude of 40° the depth of cover should be 4 to 6 feet and for 45° should be 6 to 8 feet, the smaller depths being used east of the lakes and the greater depths for the country between the lakes and the Rocky Mountains. In sandy soil the depth should be greater than in clay.

70. Special Details. To enable a pipe line to be readily inspected and repaired, stop valves should be inserted at intervals of 1 or 2 miles, and especially at important depressions and summits.

Otherwise to empty and refill a long conduit would require several days. Valves of all kinds and designs are furnished by various special manufacturing concerns.

At every summit of a pipe line ana at shut-off valves there should be placed an air valve to permit the escape of air on filling, the entrance of air on emptying, and frequently the escape of air which may gradually accumulate at summits. At all depressions, blow-off valves should be provided, the waste pipes from which should be led to a sewer, stream, or drainage channel. These valves need be only about one-third the size of the main pipe. Check valves should be introduced at points where a breakage would permit a large loss of water by backward flow, such as at the entrance to reservoirs, at the foot of long upward inclines, and in force mains just beyond the pumps. Safety valves, or pressure-relief valves, are occasionally used at the ends of long pipe lines or wherever water hammer is especially to be feared. They are simple disk valves opening outwards and held in place by springs which are adjusted to the water pressure.

The upper end of a gravity pipe line is usually enclosed in masonry and provided with a sluice gate or valve. At this point it is also desirable to have a weir or measuring sluice. The lower end of a pipe line usually terminates in a reservoir, where again valves are provided and where connections may also be made directly with the pipe system.

In crossing under other structures, such as railways, buildings, sewers, etc., special precautions should be taken to avoid all danger of future breakage.

Streams are crossed either on bridges, or by laying the pipe beneath the stream bed, or by the use of a subway.

In this country the common practice in crossing a stream is to lay a cast-iron or steel pipe below the stream bed, or else to employ a bridge crossing. Where no bridge already exists the former will ordinarily be the cheaper; and in many cases, as in navigable channels, a bridge could not be permitted. In other cases it may be cheaper to build a bridge especially for this purpose. At the angles at ends of bridge and submerged crossings special care is necessary to keep the pipe from separating at the joints. If the pipe line crosses an existing bridge, it will usually be convenient to support it beneath

the flooring. Where a bridge is built for the purpose, no floor system is put in, but merely suitable straps or stirrups to support the pipe.

The amount of protection required to prevent freezing on bridges, or at other exposed places, depends upon the size of pipe, the amount of circulation during periods of minimum flow, the temperature of the air and the water, and upon the length of the exposed portion.

Small lines, especially distributing mains, require protection. This is usually furnished by placing the pipe in a wooden box and filling around it with some non-conducting substance, such as sawdust, mineral wool, asbestos, hair felt, and the like. A mixture of plaster of Paris and sawdust has been used with good results. Any packing to be effective should be kept dry. The packing is often arranged to give one or more dead air spaces around the pipe to aid in preventing radiation.

Various methods are employed in laying pipes beneath watercourses. In the case of small streams the usual method is to employ a cofferdam and lay the pipe as on dry land. Where the water cannot readily be excluded in this way the pipe must either be put together before lowering in place or must be laid by divers. Submerged pipe should, as a rule, be laid in a trench and carefully covered to prevent injury by waves, drift ice, boats, etc.

Various special details are used in submerged-pipe laying, such as the various forms of flexible joints to enable the pipe to conform to the grade of the trench, and special joints for easy connection where divers are employed. Submerged pipe should be thoroughly tested either in sections before laying, or better, after the line is completed, in which case compressed air can be used for the purpose. Leakage of air will be indicated by the appearance of bubbles, and the imperfect joints can then be calked by divers. The various methods of laying submerged pipe will now be described together with some of the special details used in this work.

(1) Where the stream is shallow, a common method of laying is first to connect the entire pipe, or large sections of it, on platforms extending across the stream, and to lower the portion so connected by means of screws. Ordinary joints can usually be employed and the pipe put together to fit the profile of the trench. Pipes can very conveniently be laid in this way from the ice during winter.

Two cases of this method of laying will be briefly noted. At Cedar Rapids, Ia., 600 feet of 16-inch pipe was laid in this way in

a depth of $2\frac{1}{2}$ feet of water. A trench 2 feet deep was first excavated, and framed trestle bents set up 12 feet apart. A barge was then run between the legs of the trestles, the pipe put together on the barge and then slung by straps fastened to $1\frac{1}{4}$-inch threaded rods suspended from the trestles. When the entire pipe line was connected, it was all lowered together, electric-bell signals being used to secure simultaneous action among the several men stationed at the screws. The cost of laying was \$1.25 per foot.

(2) Instead of connecting the entire pipe line and lowering all together, it may be lowered in sections by the aid of flexible joints, each section consisting of several lengths of pipe connected in the usual manner. The pipe can thus be laid and lowered from a short piece of trestle or from a barge. This method is especially suitable for deep water where trestles cannot readily be used.

(3) Many lines of submerged pipe have been laid by joining several lengths on shore, towing them into position, sinking them and connecting them by divers. This method is especially applicable for large pipe lines. It has been used for large intakes at Syracuse and at Milwaukee; also at Galveston, Nashville, Boston, and many other places.

71. Cost of Pipe Lines. The cost of pipe lines will vary greatly according to the cost of the material used. This element can readily be ascertained at any time by reference to current price lists, and the item of transportation can also be quite readily determined. Cast-iron pipes laid under average conditions will cost approximately as follows, assuming the pipe itself to cost $1\frac{1}{2}$ cents per lb

Size of pipe.	Cost per foot.
4 inch	\$.50
6 "	.70
8 "	1.00
10 "	1.30
12 "	1.70
16 "	2.50
20 "	3.50
24 "	4.50

THE DISTRIBUTING SYSTEM.

Distribution Reservoirs.

72. Use. The rate at which water is actually used is not at all uniform, as fully pointed out in section 6. It varies from day to day according to the season, from hour to hour according to the time of day, and at times of large fires the rate will be greatly increased. If all parts of a system were to be designed of a capacity equal to the greatest possible rate of demand the cost would frequently be prohibitive, and in most cases it would not be the most economical plan. It will usually be more economical to store up a quantity of water in a small reservoir or elevated tank which may be drawn upon when the demand is excessive and thus relieve to some extent a part of the system.

For example, where the water is brought from the source through a long conduit, a distributing or equalizing reservoir will enable the conduit to be operated at a comparatively uniform rate and hence to be made of minimum size. Likewise such a reservoir will make it possible to reduce the capacity of pumps, or filters, or other similar works, and to operate them more uniformly and economically; or in the case of small works to operate the pumps at full capacity for a portion of the day only. In the case of a ground-water supply, a small reservoir will greatly increase the capacity of the source by making the demand more uniform. Again, in a large distributing system, several reservoirs placed at different points will effect considerable economy in the size of the pipe system. As a measure of safety against the interruption of the supply from accidents to conduit or machinery, distributing reservoirs are of great value.

In discussing forms of construction, reservoirs may be classified, according to the material employed, into (1) earthen reservoirs, (2) masonry reservoirs, (3) iron or steel reservoirs, and (4) wooden reservoirs. The first two kinds can conveniently be considered together, as the two materials are very often combined in the same structure. The last two will also be treated under the general title of standpipes and tanks.

When the reservoir does not need to be elevated above the natural surface, the most economical form, and the usual one for large capacities, is the open reservoir with earthen embankments. Such reser-

voirs are usually built with masonry walls, and covers partly in excavation and partly above the surface. If a reservoir requires to be considerably elevated, a steel standpipe or a tank of wood or steel is usually employed.

73. Capacity. Where it is possible to construct an inexpensive open reservoir at a suitable elevation and in a good location, it should be given a capacity of several days' supply. In practice the capacity of such reservoirs varies from 2 or 3 days' supply up to 8 or 10 days, and occasionally more.

Where, owing to the character of the topography, it becomes necessary to artificially elevate a reservoir in the form of a standpipe or elevated tank, the expense of construction becomes so great that the economical capacity is usually less than that mentioned under (1). The best capacity in this case depends much upon the size of the city. For large cities it is hardly practicable to provide much storage by means of artificially elevated reservoirs, the small standpipes which are often used in such cases serving merely to equalize the action of the pumps.

In small cities (up to a population of 50,000 or more) it is desirable to provide a small storage even at considerable cost, as a measure of safety and economy. The fire rate is here the principal consideration, and the minimum capacity should be such as to provide water at the maximum fire rate for a sufficient length of time to enable the pumping station to respond with ease and certainty. This is ordinarily taken as about one hour. Beyond this it will usually be desirable to add to the capacity enough to equalize the ordinary flow over several hours of the day, or, in the case of small works, to enable the pumping to be done by operating a part to the day only.

In general the best size of elevated tank will range from about 75,000 gallons for very small works up to 200,000 or 300,000 gallons for towns of the size mentioned above.

74. Location and Arrangement. The location of an elevated reservoir is governed in the first place by the topography, and the choice of location is therefore often very limited. In general a distributing reservoir should be located as centrally as possible with respect to the district to be served, as this will insure the most uniform and the highest pressures and will give the smallest size of main and branches.

In a gravity system the conduit is terminated at a reservoir, and if this reservoir is centrally located a longer conduit will be required than if it be placed near one side of the system. A proper balance must be struck between the two extremes. In a pumping system the pumps are usually located near one side of the city, and the reservoir is placed either in the vicinity of the pumps or at a more remote point in the system. In the first case all the water is usually passed through the reservoir, and the action of the pumps is very steady and uniform. In the second case a main usually leads to the reservoir from some point of the distributing system. The pumps force water directly into the system, and the reservoir takes only the surplus at times of low consumption and distributes it at times of high consumption. Certain portions of the area are thus served direct, and others are served from the reservoir. With this arrangement a more uniform pressure will be maintained in the mains, but the operation of the pumps will not be as uniform.

The proper elevation of a reservoir depends on the required pressure in the mains, a subject fully discussed in section 89.

EARTHEN AND MASONRY RESERVOIRS.

75. Form and Proportion. Earthen reservoirs are usually constructed partly by excavation and partly by the building up of embankments. If masonry walls are used in place of embankments, or as interior linings, the reservoir may be called a masonry reservoir. For single reservoirs the form most economical of material is the circular, but for large reservoirs the rectangular form is more convenient to construct and requires less land area. In practice the depths vary from 12 to 18 feet, for small covered reservoirs holding one million gallons or less, to 25, 30, or 35 feet, for open reservoirs holding 50 or 100 millions, depending upon local circumstances.

76. Construction. The construction of the embankment is based on the same principles as discussed in section 47, but the conditions are somewhat different from those obtaining with impounding reservoirs. In this case the foundation is frequently pervious and the embankment cannot be connected with an impervious stratum below. Under such conditions it is necessary to construct a water-tight lining over the entire area, and to carefully connect it with the water-tight portion of the embankment. Where a lining

is not necessary to secure imperviousness, one is usually put in to facilitate the cleaning of the reservoir.

According to circumstances the entire embankment may be impervious, or imperviousness may be secured by a puddle or concrete core, or by a layer of puddle placed near the face. Fig. 31 shows a puddle being placed near the face and Fig. 32 shows a puddle core connected to the puddle lining of the bottom.

Imperviousness is usually secured in large masonry reservoirs by a layer of puddle placed back of the wall and thoroughly rammed, and the bottom lining is treated in a similar way. In small reservoirs more reliance is placed upon impervious masonry, made so by an asphalt coating, or, more commonly, by a coat of Portland-cement

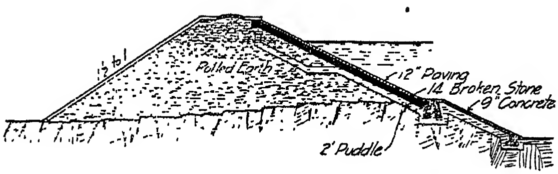


Fig. 31. Section of Reservoir Embankment, Pittsburg.

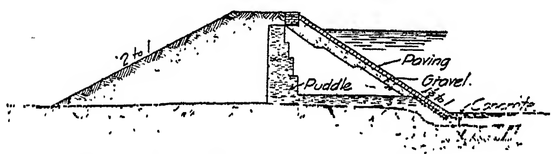


Fig. 32. Section of Reservoir Embankment, Brooklyn.

mortar, neat or 1 to 1, which it is well to finish by a brush coat of neat cement. The latter method is more likely to be satisfactory with covered reservoirs, where the temperature changes are small, than with open reservoirs.

While it is comparatively easy to secure imperviousness at the start by the use of cement, it is difficult to prevent the formation of slight cracks. These permit the water to find its way into the surrounding soil, and when the reservoir is quickly emptied this water exerts a back pressure on the walls and an upward pressure on the floor. The foundation for the walls should be broad enough to reduce settlement to very small limits, and as further precaution against cracks the floor lining should be constructed after the walls

are complete, but should be thoroughly bonded thereto. Junctions between floors and walls are preferably made curved.

The most common form of lining consists of about $1\frac{1}{2}$ to 2 feet of puddle protected by a layer of concrete, brick, or stone paving, or sometimes only by gravel. On the slopes the concrete is usually covered with paving or replaced entirely by it, experience showing that unprotected concrete is apt to be injured by ice. A layer of paving brick laid in cement makes a good finish for a concrete lining which is to be frequently exposed. Concrete can be made impervious by plastering with cement mortar, neat or 1 to 1, but where it extends over large areas, cracks will form, due to temperature changes and to settlement of embankments. To minimize this difficulty, concrete may be laid in blocks, with asphalt joints between.

If ground water is met with, which is under considerable pressure, it will be necessary, in order to avoid rupture of the floor, to drain the soil beneath the lining. In some cases the ground water has been permitted to enter the reservoir, when its head exceeds that in the reservoir, through flap valves which will close when the difference of head is in the reverse direction. Drainage of the soil beneath the lining should be done with great caution, and especial care taken to surround all drains with gravel and sand so graded in fineness as to effectually prevent the washing out of any of the material. Seepage water is also sometimes taken care of by means of drains.

Asphalt has for some time been extensively used for reservoir linings on the Pacific coast, and recently its use has become quite general. Compared to concrete it has the advantages of elasticity and greater imperviousness, both of which are of great importance in this connection. Another advantage in many cases is its cheapness. Its chief disadvantage is the effect of the sun in rendering it more or less plastic and liable to creep if used on steep slopes. Its durability in water is also not fully determined. Great care and expert knowledge are required in determining the proper proportions of the various ingredients to give good results.

When the earth is firm and compact, asphalt linings can be placed directly upon it, and have frequently been so placed. Considerable settlement has in some cases taken place without cracking the lining, but this cannot, of course, be relied upon.

Reservoirs with masonry walls occupy less space than earthen reservoirs, but are more expensive to construct. They are, however, often the best form for small reservoirs where space is limited, and are a suitable form in case covers are required.

When the reservoir is excavated in firm earth or is backed by a well-compacted embankment, the earth serves to support the walls against water pressure. They must then be designed to sustain the earth pressure with reservoir empty. By adopting the circular form the masonry will resist largely by compression as a ring, and the dimensions can be considerably reduced below those required for a wall resisting by gravity alone. Several small circular reservoirs have been built of diameters of 50 to 75 feet, with walls from 16 to 22 inches in thickness.

The masonry may be of rubble, concrete, or brick, according to circumstances. If exposed, a lining of paving brick makes an excellent finish. It is needless to say that in all work of this character the greatest care should be taken to secure the best workmanship, particularly in the mixing and laying of concrete and the thorough filling of masonry joints with mortar, essentially as in dam construction.

77. Inlet Pipes and Valves. Distributing reservoirs are usually provided with separate inlet and outlet pipes, located preferably on different sides of the reservoir in order to promote circulation of the water. In earthen reservoirs these are constructed in the same manner as described in section 48. A by-pass should be provided to enable the reservoir to be cut out at any time. Where the reservoir serves merely as an equalizing reservoir, receiving only the surplus water from the distributing system, a single pipe will serve for both inlet and outlet.

In open masonry reservoirs gate chambers are conveniently built in connection with the reservoir wall. In covered reservoirs they are usually omitted, the valves being placed within the reservoir and operated from a suitable platform or from the oustide.

78. Covered Reservoirs. Ground waters should be stored in covered reservoirs, for the reason that such waters usually contain sufficient quantities of plant food to promote a luxuriant growth of vegetable organisms unless the light be excluded. Many cases have arisen of bad tastes and odors due to this cause which have been

entirely removed by covering the reservoir. Filtered surface waters should also as a rule be stored in covered reservoirs, since by the process of filtration they are rendered somewhat similar in nature to ground waters. Where reservoirs are located in the densely populated portions of cities, covers are also advisable, in order to exclude soot and dust.

Covers are usually made of masonry, but wood has been used in a number of cases. It is much cheaper than masonry, but is much less durable and does not keep the water as cool in summer or wholly prevent freezing in winter.

A wooden cover for a large area may consist simply in a horizontal floor of boards, supported by a system of joists and girders resting on a series of wooden posts. For small areas the covers can readily be made sloping, and this is a preferable arrangement. Covers for small circular reservoirs and large wells are conveniently made conical, with the rafters resting against the wall or supported on light trusses.

Masonry covers consist usually of segmental or elliptical masonry arches supported by small brick piers; or, for very small reservoirs, a dome may be used. Above the arches, about 2 feet of earth is placed to prevent extreme variations of temperature and to protect the masonry, and embankments are constructed against the side walls to meet the covering above. The piers are spaced from 10 to 15 feet apart, and are made from 1 to 2 feet square in cross-section, depending upon the span and weight of filling.

Piers are usually made about 18 inches square of brick and spaced about 12 to 15 feet apart. Concrete is now generally used for the roof, being made in the form of groined arches of about 3 feet rise and 6 inches thick at the crown. Fig. 33 shows the interior of such a reservoir used as a filter.

79. Cost. The cost of reservoirs varies, of course, greatly according to local conditions, kind of reservoir and capacity. According to the capacity the cost per unit will be less the larger the reservoir. The actual cost of a large open reservoir varies from \$3 to \$5 per 1,000 gallons capacity. Covered masonry reservoirs will cost usually from \$10 to \$15 per 1,000 gallons capacity.

STANDPIPES AND ELEVATED TANKS.

80. Where a reservoir requires to be artificially elevated it is usually built as a standpipe—a tall slim tank resting on the ground—or as an elevated tank of steel or wood, supported by a tower of steel, wood, or masonry. Such an elevated reservoir may or may not be enclosed in a covering of masonry or wood, according to the necessities of the case and the notions of the designer.

Reservoirs of this type are relatively so expensive that a minimum amount of storage capacity is usually provided. As shown

Fig. 33. Covered Filter.

in section 72, they may be used in small towns to enable the pumps to be more economically operated, or in larger towns to provide for fire consumption for an hour or so. The capacities of standpipes and tanks range ordinarily from 50,000 gallons up to a maximum of about 1,500,000 gallons for small villages and cities up to 30,000 population or more. The useful capacity of a standpipe is only that part of the volume which is at a sufficient elevation to give the required pressure. All water below this level acts merely as a support for the portion above. There should therefore first be deter-

mined the lowest useful level of the water, and the pipe should then be made of the desired capacity above this plane.

81. Location. For storage purposes only, the location would be the same as that for any reservoir. To reduce the cost it is, however, desirable to place the tank on the highest ground available if it be within a reasonable distance. Too great distances will be undesirable on account of the cost of mains and the loss of head caused by a long line of pipe.

82. Design of the Standpipe. The chief elements in the design of a standpipe are the thickness of plates, the riveting, the foundation and anchorage and the pipe details. The forces to be considered in the design of a standpipe are the water pressure, the wind pressure, the weight of the pipe, and the action of ice. In what follows let h = distance in feet of any point of the pipe from the top, d = diameter of pipe in feet, r = radius in feet, and t = thickness of shell in inches at the given point.

From equations in Hydraulics we find that the water pressure causes a bursting stress per vertical lineal *inch* of pipe equal to

$$S = \frac{62.5hd}{2 \times 12} = 2.6hd \qquad (9)$$

The stress per square inch of metal is

$$s = \frac{2.6hd}{t} \qquad (10)$$

This is the only stress that need be considered in determining the plate thickness, as the effects of wind and weight are much smaller than this and cause a stress in a *vertical* direction.

The safe tensile stress on net section of metal, where but little ice is likely to form, may be taken at about 15,000 pounds per square inch. Where thick ice is to be expected the working stress should be reduced to 12,000 or even 10,000 pounds, to provide for the unknown ice stresses. The vertical joints will usually be so designed as to have an efficiency of 60 to 70 per cent. If a = safe stress on net section and e = efficiency, then by equation 10 the required thickness to resist the water pressure will be

$$t = \frac{2.6hd}{s} = \frac{2.6hd}{ae} \qquad (11)$$

or, if $a = 12{,}000$ and $e = \frac{2}{3}$, then, approximately,

$$t = \frac{2.6hd}{8{,}000} = .000325hd \qquad \textbf{(12)}$$

The thickness near the top should not be less than $\frac{1}{4}$ inch, or for very large pipes, $\frac{5}{16}$ inch. Plates thicker than 1 inch or $1\frac{1}{8}$ inches should be avoided.

The plates forming a standpipe are usually of such a width as to build 5 feet of pipe, and are from 8 to 10 feet long. Each course is preferably made cylindrical, and alternately an "inside" and an "outside" course.

The riveting of the vertical seams is the most important part of the construction, as this determines the strength and economy of the standpipe. Lap joints are most commonly used, but for thickness exceeding $\frac{1}{2}$ inch, double-butt strap joints are much preferable and are stronger.

Table No. 12 gives suitable proportions for riveted joints, together with their approximate efficiencies or ratio of strength of joint to strength of plate.

TABLE 12.

Proportions for Riveted Joints for Standpipes.

Kind of Joint on Vertical Seams.	Thickness of plate.	Diameter of rivet.	Pitch of rivets. Centre to centre.	Distance between Pitch-lines.	Distance of Pitch-line from edge of plate.	Efficiency, per cwt.
	Inch.	Inches.	Inches.	Inches	Inches	
Single-riveted lap	$\frac{1}{4}$	$\frac{5}{8}$	$1\frac{5}{8}$	.....	1	50
" " "	$\frac{5}{16}$	$\frac{5}{8}$	$1\frac{3}{4}$	.....	$1\frac{1}{8}$	
Double " "	$\frac{5}{16}$	$\frac{5}{8}$	$2\frac{5}{8}$	$2\frac{1}{8}$	$1\frac{1}{8}$	60
" " "	$\frac{3}{8}$	$\frac{3}{4}$	$2\frac{5}{8}$	$2\frac{1}{8}$	$1\frac{1}{4}$	
" " "	$\frac{7}{16}$	$\frac{3}{4}$	$2\frac{3}{4}$	$2\frac{1}{4}$	$1\frac{3}{8}$	
" " "	$\frac{1}{2}$	$\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{1}{2}$	
" " butt	$\frac{9}{16}$	$\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{5}{8}$	70
" " "	$\frac{5}{8}$	$\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{5}{8}$	
" " "	$\frac{11}{16}$	1	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{3}{4}$	
" " "	$\frac{3}{4}$	1	3	$2\frac{1}{2}$	$1\frac{3}{4}$	
" " "	$\frac{13}{16}$	1	3	$2\frac{1}{2}$	$1\frac{7}{8}$	
" " "	$\frac{7}{8}$	$1\frac{1}{8}$	3	$2\frac{1}{2}$	2	
" " "	$\frac{15}{16}$	$1\frac{1}{8}$	3	$2\frac{1}{2}$	2	
" " "	1	$1\frac{1}{4}$	$3\frac{1}{8}$	$2\frac{5}{8}$	$2\frac{1}{8}$	
Triple " "	1	$1\frac{1}{8}$	4	3	$2\frac{1}{8}$	75

Horizontal joints are made single-riveted lap joints, with rivet spacing of about three diameters. All seams should be thoroughly calked with a round-nosed calking tool, and any leaky seams which may exist when the pipe is filled should be recalked. The bottom is made of plates riveted up with circular and radial joints, the former being made lap joints and the latter butt joints. The thickness need be only enough to permit of good calking and to be durable,—about $\frac{1}{2}$ inch. This bottom plate is preferably connected to the side plates by means of a heavy angle on the outside, or one on both outside and inside the tank. The foundation should be made monolithic and sufficiently broad to give such low pressures on the soil that there will be practically no settlement. Failures have occurred due to poor work in this respect. Wind pressures should be carefully considered. Concrete is a very suitable material for foundation purposes.

Standpipes must be anchored to the foundation to prevent being overturned by the wind. The wind pressure is usually taken at 40 to 50 pounds per square foot on one-half the vertical projection of the tank. At the higher value the overturning moment in foot pounds at a distance h below the top is

$$M = 50 \times \frac{dh}{2} \times \frac{h}{2} = 12.5dh^2 \qquad (13)$$

This movement causes an uplift on the leeward side for each inch along the circumference of the pipe of

$$S^* = 1.33\,\frac{h^2}{d}\text{ pounds.} \qquad (14)$$

Then if anchor bolts are placed p inches apart around the bottom of the tank the stress in each bolt will be

$$S \times p = 1.33\,\frac{h^2}{d} \times p \qquad (15)$$

If numerous bolts are used, their size need not be great, and they may be put through the exterior bottom angle iron and the latter double-riveted to the pipe. If arranged in this way, they should be numerous enough so that the stress in one bolt is not greater than can

*The derivation of this equation comes from the formula $S = \frac{Mc}{I}$ of Mechanics in which I is the moment of inertia of the standpipe shell. The process of derivation cannot well be entered upon here.

be transmitted to the lower plates by four or five rivets, which will limit the size of bolts to about $1\frac{3}{4}$ times the diameter of the lower rivets. By spacing the bolts sufficiently close this arrangement may be followed in almost any case. If this method gives a large number of bolts, it will be simpler to use fewer and larger bolts, in which case they should be fastened to the standpipe by long vertical pieces of angles, and the bolts placed close to the pipe as shown in Fig. 34. The number of bolts should not be less than six in any case. Anchor bolts should extend well into the masonry and be fastened to anchor plates embedded therein.

Besides the overturning effect of the wind there is to be considered the collapsing effect on the empty pipe, especially near the top where the plates are thin. This cannot readily be computed, but must be provided for by an ample margin of strength at the top of the standpipe.

The effect of ice action is a very serious matter in unprotected standpipes, but is very difficult to calculate or provide for. The stresses caused by ice action can only be provided for by the use of a good quality of soft steel which will allow of deformation without injury, and by the use of a large factor of safety. It may well be questioned, in view of the uncertainties of the case, if all metal tanks built in cold climates should not be encased in masonry or wood. The importance of this matter is attested by the many accidents traceable to the action of ice.

The material used for standpipes should be soft, open-hearth steel, of a tensile strength of about 54,000 to 62,000 pounds per square inch. The best practice now calls for a grade corresponding to flange steel, with phosphorus limit of about .06 per cent, an elongation of 22 to 25 per cent, reduction of area of 50 per cent, and flat bending tests, both cold and after heating and quenching.

83. Pipes and Valves. Usually a single pipe serves both as inlet and outlet. This passes through an arched opening in the foundation, turns upwards and enters the standpipe at the bottom, and extends into it a foot or two. A lead joint is usually made in a bell casting riveted to the bottom of the pipe as shown in Fig. 34. A drain-pipe through which the tank may be drained or flushed should also be provided.

High-water electric alarms are advisable if the pipe be at some distance from the pumping station. The pressure indicated at the

station is not a certain guide if branch mains are led off at intermediate points. For encased pipes or tanks a simple float, arranged to close an electric circuit, may be used. For exposed pipes, ice is likely to interfere, and in this case a pressure gauge placed in a vault and connected to the standpipe can be arranged to give an alarm at any desired pressure.

84. **Other Details.** The top should be stiffened against collapse by a heavy angle-iron, not less than 3 × 5 inches, and two such angles should be used for large pipes. The effect of the wind on an empty pipe is not only to cause a pressure on the outside, but to create a partial vacuum on the inside near the top. Several failures have occurred from lack of strength at this point.

It is not customary to roof standpipes, and for a tall slim pipe a roof would be of little use and no improvement to its appearance. With large, low pipes a conical roof of curved profile may well be adopted. It affords considerable protection and improves the appearance of the structure. It is usually made of sheet iron or copper, supported on light angle-iron ribs or a framework.

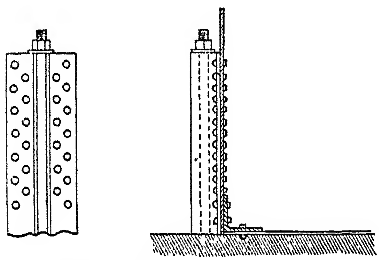


Fig. 34. Arrangement of Rivets.

A ladder should be built on the outside of the pipe, but none on the inside; and in general there should be no obstructions on the inside where ice is likely to form to any extent.

Standpipes should be well painted inside and out. For the interior, asphalt is probably the best material to use. After painting the interior, the pipe should be filled to detect leaks before the outside is coated.

A standpipe is often surrounded with a masonry shell in order to furnish protection from cold, or to improve the appearance of the structure. This shell may be of stone or brick, and is usually built enough larger than the pipe to permit of a stairway in the space between. The walls are usually made from $2\frac{1}{2}$ to 4 feet thick at the bottom and $1\frac{1}{2}$ to 2 feet at the top.

Encased pipes must be provided with overflows, which may be built either inside or outside the main pipe. For this type of struc-

ture, roofs are quite necessary, and should be carefully proportioned with respect to appearance. The masonry offers considerable opportunity for architectural treatment, and this feature should be referred to a competent architect.

85. Elevated Tanks. If the lower portion of the water in a standpipe is at too low an elevation for useful pressure, its only office is to furnish support to the useful part above. Where this useless zone is of any considerable depth the support can be more cheaply furnished by a steel trestle. Besides being cheaper, a tank is much less objectionable in appearance than a standpipe, and experience indicates that trouble from ice is less likely to occur. For roofed tanks a height equal to the diameter would not be far from the most economical proportions, but a height somewhat greater than this will usually look better.

The bottom of a tank supported on an iron trestle is usually made hemispherical, as this requires no support except at the outside edge where the legs of the tower are located. The thickness of side plates is the same as for standpipes, and the details are similar. If the bottom is hemispherical the stresses therein will be one-half those in the lowest side course of plates.

The tower consists of a steel trestle of four to eight legs. The material for this may be medium steel, and comparatively high working stresses may be used in its design, since the stresses are all dead- and wind-load stresses. Four legs are the smallest practicable number, but for tanks of large diameters the use of only four legs brings very heavy local stresses on the tank at the points of connection. Six or eight is a better number and presents a better appearance, but is more expensive. The stresses in the various parts of the tower and the design of the details belong to the domain of structural engineering and cannot be elaborated here. Suggestions for the upper column connections, the anchorage and the roof are given in Fig. 35.

Each column must be well anchored to the foundation, with a strength of anchorage equal to the maximum uplift due to wind acting on empty tank. The foundation should be rigid, and large and heavy enough to serve as anchorage and to give only safe pressures on the ground. There should be practically no settlement, as any unequal settlement will greatly change the stresses in the tower.

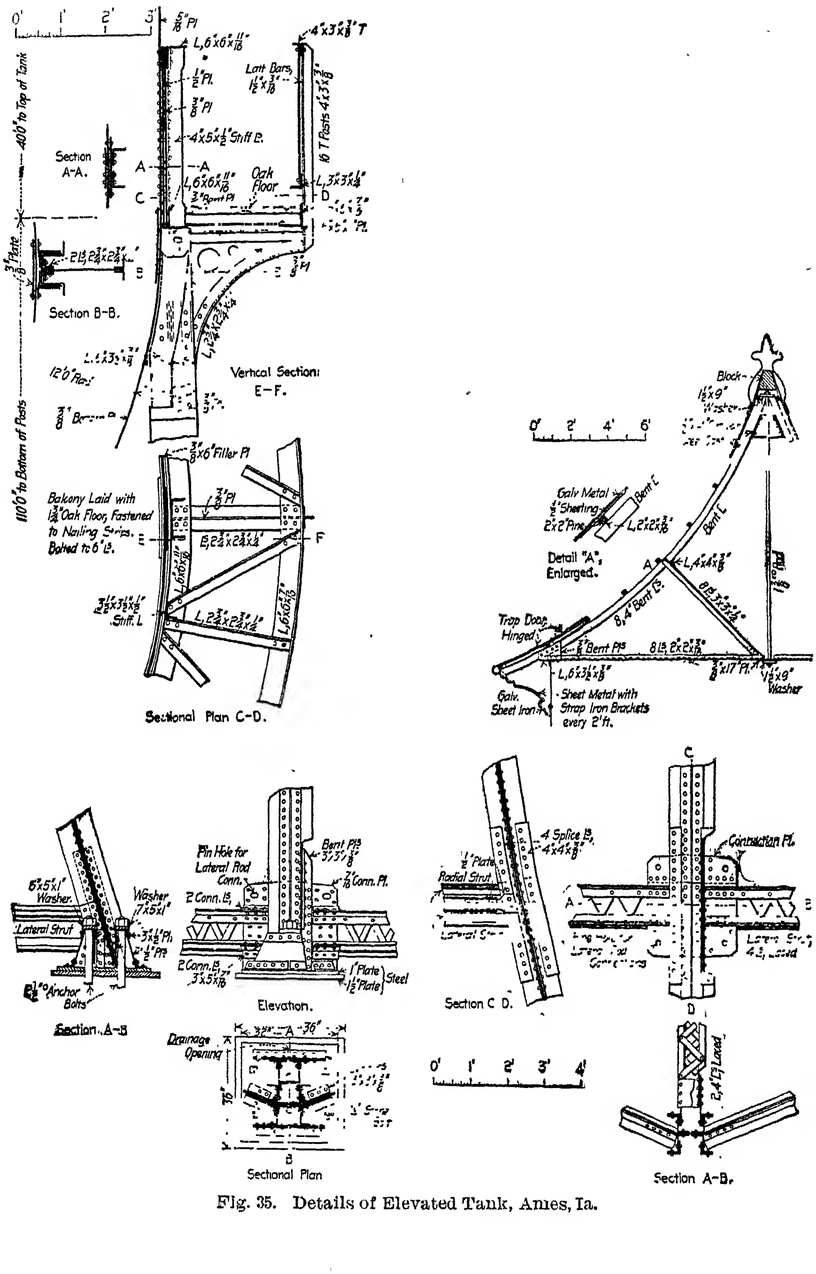


Fig. 35. Details of Elevated Tank, Ames, Ia.

The inlet pipe is usually made to enter the tank at the center of the bottom, and should be provided with an expansion joint. In cold climates the pipe must be protected by a frost casing, which is usually a simple wooden box with one or more air spaces and a packing of some non-conductive material. If the tank is encased, it will be necessary to provide an overflow pipe.

86. Wooden Tanks. Elevated tanks of wood are frequently used where low first cost is an essential element and the quantity to be stored does not exceed 50,000 to 75,000 gallons. Wooden tanks are cheap, and if well built will last fifteen or twenty years. The staves should be of good clear material and should be dressed to proper curvature on the outside. Hoops should be relatively thick to resist corrosion, and should be thoroughly coated with asphalt or other protective coating, before being put in place. Lugs and fastenings are a source of weakness. They should be carefully designed and of ample strength. The support of the floors must also be well looked after. The chief source of trouble with wooden tanks is in the weakening of the hoops by rusting from the inside.

Several failures of wooden tanks have occurred by the sudden bursting of the hoops, and it is questionable policy to construct such tanks where their failure is likely to endanger life, as it is quite certain that they will not be regularly inspected as they should be.

87. Storage Under Compressed Air. In small works, air chambers or their equivalent may be used to provide a considerable storage of water and thus avoid the use of standpipes or elevated tanks. In the design of such storage tanks the larger the proportion of air space the less will be the variation in water pressure as the tank is emptied. If V = volume of tank, and v = maximum volume of water stored, then $V - v$ = minimum volume of air. If the pressure, when containing the maximum volume of water, be P, then when the tank is just empty the pressure is $p = P\left(1 - \frac{v}{V}\right)$. Thus if $\frac{v}{V} = \frac{1}{3}$, then $p = \frac{2}{3}P$, and the variation in pressure is one-third the maximum. The less the desired variation in pressure the greater must be the tank capacity for a given water capacity. The air is maintained in the tank by occasionally admitting a little air into the pump.

A system of pressure storage having several advantages over that just described is the Acme Company's system, based on patents of Wm. E. Wortham and Oscar Darling. In this system the air is stored in a separate tank at a higher pressure than is ordinarily kept in the water tank. By reducing valves in the connecting pipes, the pressure on the water may be maintained constant, or may be increased in case of fire up to the pressure in the air tank. Air compressors must be used here to keep up the air supply. A number of plants of this kind have been installed. The use of a pressure storage system avoids all trouble from ice, and for very small quantities is cheaper than an elevated tank. A storage tank can also be located at the pumping station and the pressure easily controlled. For large quantities the system would be very expensive.

THE DISTRIBUTING PIPE SYSTEM.

88. General Requirements. A distributing system should be so designed that it will be able to supply adequate quantities of water to all consumers, and that this will be accomplished with economy and with reasonable security against interruption. With respect to the design of this part of a waterworks system, the uses of water naturally fall into two very distinct classes: (1) the ordinary, everyday use for domestic, commercial, and public purposes; and (2) the use for fire extinguishment. In the former case the consumption is relatively uniform over different portions of the city, and is also well distributed over many hours of the day; in the latter case the rate is likely to be extremely high for a very short period of time, but this excessive use of water will usually be confined to a comparatively small area. To supply water in the former case requires the wide distribution of moderate quantities, while in the latter case the problem is rather the concentration of large volumes within a narrow district, which district may be situated at any point in the system.

To supply water to all consumers requires that a pipe be laid in each street, except in those cases where the cross streets are not built upon. In the outlying districts, pipes are laid in those streets where the density of the population warrants it, according to the judgment of the management, but much difference in policy exists in respect to the matter of extensions. The distributing system includes, besides

the pipes, the fire hydrants, service connections, valves, fountains, watering troughs, meters, and occasionally other details.

89. **The Pressure Required.** For ordinary service the pressure at any point should be sufficient to supply water at a reasonable rate in the upper stories of houses and factories, and in business blocks of ordinary height. This will require at the street level a pressure of from 20 to 30 pounds in residence districts, and usually from 30 to 35 pounds in business districts, according to the character of the buildings.

For fire purposes the pressure required in the mains depends upon whether it is intended that fire streams shall be furnished directly from the hydrants or whether steam fire engines are to be used. In small cities and towns it is of the greatest advantage to supply fire streams without the use of engines, and in most such places this method is adopted, fire engines being sometimes kept in reserve, for extraordinary conflagrations. In pumping systems the most common arrangement is to maintain only a moderate pressure for ordinary service, and at times of fires to shut off the reservoir or standpipe if there be one, and to furnish the necessary fire pressure direct from the pumps.

In large cities hydrant fire pressure is not so common, but if the supply is by gravity, and has plenty of head, a hydrant fire pressure can profitably be furnished, at least for all except the densest portion of the city or for very large fires. If hydrant fire pressure is to be supplied it should not be less than 60 pounds for residence districts and 70 pounds for business districts. Pressures 20 pounds higher than these are to be desired. If dependence is to be placed on fire engines, as is usual in large cities, the domestic pressure of 25 to 30 pounds is sufficient.

The pressures here considered are the hydrant pressures at times of maximum consumption, and refer to any point in the distributing system. If such pressures are maintained at the most remote points and at the higher elevations, the pressures on the lower ground and at points nearer the pumps or reservoir will of course be considerably higher. In the case of gravity supplies much higher pressures may be possible, but on account of the increased cost of plumbing and piping to withstand high pressures it will not be desirable often to exceed 130 to 140 pounds.

90. Number and Size of Fire Streams. The number of fire streams which should be simultaneously available in any given town will obviously vary greatly with the character of the buildings, width of streets, etc. For average conditions the number may be calculated from the formula

$$y = 2.8\sqrt{x}, \qquad \textbf{(16)}$$

where y = number of streams, and x = population in thousands. About two-thirds of this number should be capable of being concentrated upon a single block or group of buildings.

In small cities and towns the requirements for fire protection may differ widely. For example, in a country town of 4,000 to 5,000 inhabitants, in which only a small mercantile business is carried on, the fire risk is not great, while in a town of the same size whose prosperity depends entirely upon two or three large factories, located, perhaps, in one large group of buildings, a fire would be a very serious matter. In the former case four or five fire streams would be sufficient, while in the latter case eight or ten should be supplied.

The number of fire streams is based upon a size of stream of about 250 gallons per minute, which is generally considered to be about right as an average value for good fire streams in business districts. For a residence district 175 to 200-gallon streams will usually meet the requirement. Fire hydrants must be sufficiently numerous and so located as to meet the requirements regarding number and size of fire streams set forth in the preceding paragraphs. Hydrants are one-way, two-way, three-way, etc., according to the number of hose connections provided. For most purposes the two-way hydrant is considered the most convenient, but in the dense portion of a large city, where many connections must be provided, three-way and four-way hydrants can be used to good advantage. Hydrants should, in any case, be numerous enough to enable the required number of streams to be furnished with a suitable nozzle pressure. At points where a large number of streams are required, fire cisterns are sometimes used instead of hydrants. These cisterns are fed by large pipes, and have an advantage over hydrants in that they allow several steamers to obtain their supply at one point.

For a 250-gallon stream the required nozzle pressure is 45 pounds and the loss of head per 100 feet of ordinary $2\frac{1}{2}$-inch hose is about 18 pounds (see Hydraulics), so that with a hydrant pressure of 100

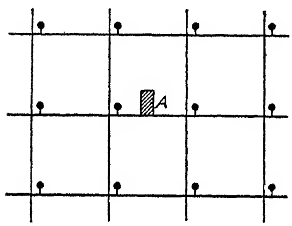
pounds the length of hose to supply a 250-gallon stream cannot exceed 300 feet. A 175-gallon stream, with a 1-inch nozzle, requires 35 pounds nozzle pressure, and causes a loss of head of 9 pounds per 100 feet of hose. With a hydrant pressure of 100 pounds the length of hose in this case might be 700 feet. With a hydrant pressure of 75 pounds, which is quite common, a 250-gallon stream could not be supplied through a length of hose greater than about 200 feet, and a 175-gallon stream through a length greater than about 450 feet. Hence the general rule that hydrants should be so spaced that no line of hose should exceed 500 to 600 feet, and for at least half of the streams required at any point the length of hose should not exceed 250 to 350 feet, according to the hydrant pressure. These lengths cannot be much increased even where fire-engines are used. In outlying districts two two-way hydrants should be available at any point, with a distance of not more than 500 to 600 feet to the more remote of the two.

The most convenient location for hydrants is at the street intersections, as they are then readily accessible from four directions. In cities of moderate size the required number of streams can readily be supplied by locating a hydrant at each street intersection, but in large cities intermediate hydrants are often necessary. Thus if the blocks in Fig. 36 are 300 feet long in each direction, and a two-way hydrant is placed at each corner, then a fire at A could be served from eight hydrants, with a maximum length of hose of about 450 feet, giving sixteen good fire streams; while a fire at a street corner could be served from thirteen hydrants, eight of which would, however, require hose lengths of 600 feet. With blocks 600 feet by 300 feet, as in Fig. 37, a two-way hydrant at each intersection would supply not less than eight streams at any point, without exceeding 600 feet of hose. If only four streams are required, then one-fourth of the hydrants might be omitted, or every other hydrant in alternate streets, as hydrants 1, 2, and 3.

91. General Arrangement of the Pipe System. From the data on page 9 it is evident that the fire demand will largely govern in the design of the pipe system. This is more and more true the smaller the town or district considered, and for single blocks the ordinary consumption can practically be neglected. To supply long, narrow districts, the general scheme would be to furnish the water

mainly through a single large pipe of gradually decreasing size, with small parallel and branch mains supplying the side streets. For broad areas, such as comprise the larger portions of most cities, the general arrangement usually adopted is to provide large mains at intervals of $\frac{1}{4}$ to $\frac{1}{2}$ mile, and to fill in between these mains with smaller pipes, thus forming a gridiron system.

A general principle which should be kept in mind when laying out a system is to so arrange the large mains that the smaller cross mains may be fed from both ends, since a pipe so fed is equivalent to two pipes. It can furnish double the number of streams with the same loss of head, or the same number of streams with about one-fourth the loss of head, as when fed from one end only. This principle also makes it desirable to lay connecting pipes between separated districts,



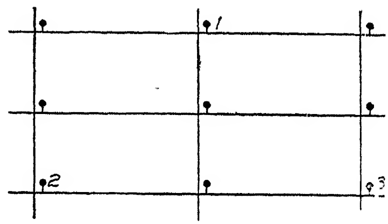


Fig. 36. Location of Hydrants. Fig. 37.

even when such pipes are not required for supplying local consumers. In the case of fire, each district may then be served from both ends. Dead ends are also objectionable on account of the stagnation which exists in the pipes and the deterioration of the water which is likely to ensue.

The size of mains and cross lines in the gridiron system will depend largely upon the number of fire-streams required at any point. In small cities, and outlying districts of large cities, 6-inch cross mains with 8, 10, or 12-inch pipes at intervals of four to six blocks is a common arrangement. Four-inch pipe should never be used to supply hydrants except where the pipe is comparatively short and is fed from larger pipes at each end.

92. Calculation of the Pipe System. For the purpose of calculating the distributing system it is necessary to know the maximum rate of consumption for the entire city, and for large and small sections of the same, with suitable consideration for future growth.

The rate for the entire city will enable the main supply conduit, or the principal force main, to be determined. For calculating the main distributing pipes the city should be divided into relatively large districts, corresponding to the most probable location of such main arteries; then for the smaller pipes the demand for still smaller sections must be considered, and so on.

The maximum rate of consumption for the entire city has already been discussed in section 6. From the data there given the ordinary maximum rate is seen to be from 200 to 250 per cent of the yearly average. If the yearly average be 100 gallons per capita daily, the maximum ordinary rate will then be about 250 gallons per capita per day, or 0.17 gallon per capita per minute. The maximum fire rate, assuming 250-gallon streams, is $250 \times 2.8 \sqrt{x}, = 700 \sqrt{x}$ gallons per minute, where x = population in thousands. Thus for a population of 1,000 the ordinary maximum rate may be about 170 gallons per minute, while the fire rate is likely to be 700 gallons, or four times as much.

After estimating the maximum rate of consumption for the city as a whole, the same should be done for the several districts, the probable future population, the maximum ordinary rate, and the maximum fire demand being estimated for each district independently. The data so collected will enable the main distributing pipes to be calculated. The size of the cross mains and smaller pipes will be determined almost entirely by the local requirement as to fire streams. For all practical purposes an arrangement of 6-inch pipe in one direction, and 4-inch pipe crossing these, is ample for cities up to about 10,000 inhabitants, and six-inch pipes in both directions will suffice for populations up to about 50,000. For villages up to 1,000 or 2,000 population and the residence districts of small cities all but the general supply main may be 4-inch provided there are no dead ends and that there is a cross line at least every other block.

The size of the main supply pipe and the main branches feeding isolated districts can be calculated by the aid of Table No. 12 of Hydraulics giving the friction loss in pipes, an estimate of the maximum rate of demand having been made. For most cases the desirable velocities in the main pipes will be from 3 to 6 feet per second for the maximum rate of flow. The lower velocity is that suitable for a plant where the available head is limited and not much

friction loss can be permitted, as for example in a gravity system where the elevation of the source is barely sufficient to furnish the desired pressure. The higher velocity is suitable where a considerable loss of head may be allowed, as for example in a gravity system with a high source, or in a pumping system where the fire pressure is furnished by pumps and only during the fire.

The number of fire streams of 250 gallons per minute each which can be supplied reasonably through pipes of different size are given in Table No. 13, the smaller number corresponding approximately to the lower velocity mentioned above and the larger the higher velocity. Where a pipe is fed from both directions double the number of streams can be supplied.

TABLE 13.

Number of Fire Streams Obtainable From Pipes of Various Sizes.

Size of pipe.	No. of 250-gal. streams.
4	1
6	1– 2
8	2– 4
10	3– 6
12	4– 8
16	8–16
20	12–24
24	18–36

Example. A town of 3,000 inhabitants is to be supplied through a force main 4,000 feet long. Assuming the average daily consumption to be 75 gallons per capita and that the town is of average character as regards fire demands, what would be a suitable size of main?

Referring to section 6, we find that the maximum rate for ordinary use may be taken at 180 per cent of the average, which would be $1.80 \times 75 = 135$ gallons. The rate per minute will be $\frac{135 \times 3{,}000}{24 \times 60} = 280$ gallons. The number of fire streams required is by formula 16 equal to $2.8 \sqrt{3} = 4.8$ or, say, 5. Each being assumed as 250 gallons the total rate will equal $280 + 5 \times 250 = 1{,}530$ gallons per minute, or practically equal to 6 fire streams. From the table No. 13 we see that a 10-inch pipe may be used if a considerable loss of head is permissible or a 12-inch pipe if but little

loss is desired. From Table 12 of Hydraulics the actual loss of head in the 10-inch pipe for a flow of 1,530 gallons per minute is 16 feet per 1000, or 64 feet for the entire length of main. For a 12-inch pipe the loss is only about 6.5 feet per 1,000, or 26 feet total. Where the available head is not more than 150 feet the former loss would be too great.

93. Separate Services for Different Elevations. Where the different parts of a town vary considerably in elevation, it is frequently advisable to divide the distributing system into two or more independent portions, each serving an area or zone situated between certain limiting elevations. It often happens that only a small portion of a city is at a high elevation, and by thus separating the systems of distribution a comparatively small amount of water will need to be raised to the maximum height, the greater portion being pumped against a much lower pressure. By this arrangement a large saving can be effected in the expense of pumping, and the use of excessive pressures in the lower districts will also be avoided.

Various arrangements may be made for supplying the different zones. Each zone may be practically an independent system, with its own pumping station and perhaps its own source of supply; or the pumps of a higher zone may be supplied by a reservoir located at a high point in the next lower zone; or the pumps of the different zones may all be located at the same station and obtain their supply from the same source. In the gravity system a division is often made so that the lowest zone is supplied by gravity, while the upper zones are supplied by pumps.

94. Location of Pipes and Valves. The distributing pipes should be so located with respect to street lines as to be readily found and to avoid other structures as far as practicable. The center of the street being usually reserved for the sewer, the water pipes are placed at some fixed distance, usually from 5 to 10 feet from the center. The side chosen should be the same throughout. The north side of east and west streets will be warmer than the south side.

Valves should be introduced in the system at frequent intervals so that comparatively small sections can be shut off for purposes of repairs, connections, etc. As a general rule, wherever a small pipe branches from a large one, the former should be provided with a

valve. Thus with 10 or 12-inch pipes feeding 6-inch pipes, each of the latter should have a stop valve at each end. At intersections of large pipes a valve in each branch is usually desirable. In a network of small pipes of uniform size, a valve in each line at each intersection, or four in all, is rather more than necessary, but two at each intersection, or a valve in each line every two blocks, answers very well.

Valves, like pipe lines, should be located systematically. They are usually located in range either with the property line or the curb line, but sometimes are placed in the cross walks.

95. Hydrants. The general location of hydrants has already been considered in section 90. In fixing upon the exact location, and the side of the street on which each should be placed, a detailed examination should be made and the location determined with reference to important buildings, convenience of access in case of fires, etc. Generally the hydrant is placed on the same side of the street as the pipe, and is connected to the larger of two pipes where there is a choice.

Hydrants are of two general types—the post hydrant, in which the barrel of the hydrant extends 2 or 3 feet above the ground surface, and the flush hydrant, in which the barrel and nozzle are covered by a cast-iron box flush with the surface. The former is more commonly used, and as it is much more readily found and more conveniently operated, it is to be preferred, except perhaps in the congested districts of large cities, or on narrow streets where all obstructions should be avoided. Post hydrants are set just back of the curb line; flush hydrants, either in the sidewalk or in the street.

Many styles of hydrants are on the market, most of which will give reasonably good service if properly handled. Reliability of operation is the first essential, but next in importance is the requirement that the frictional loss in the hydrant shall be small. All waterways should be ample, and sharp angles and sudden changes in size should be avoided as much as possible. Considerable difference exists in different hydrants in this respect, with a corresponding difference in the amount of pressure lost. In Fig. 38 are shown two forms of hydrants which illustrate the two general types of valves used—the gate valve and the compression valve. In ordering

hydrants care should be taken to have the nozzles of the same standard as those used in adjoining large cities, so that connections can readily be made to fire apparatus which may be borrowed in emergencies.

When a hydrant is closed after use, the water remaining in the barrel must be drained out through a drip, so arranged as to open when the main valve is closed. This is an important feature of the design, as a hydrant is likely to freeze if not thoroughly drained. The escaping water may be led away through a small drain pipe to a sewer, or a considerable body of broken stone and gravel may be filled around the base, into which the water may be allowed to drain.

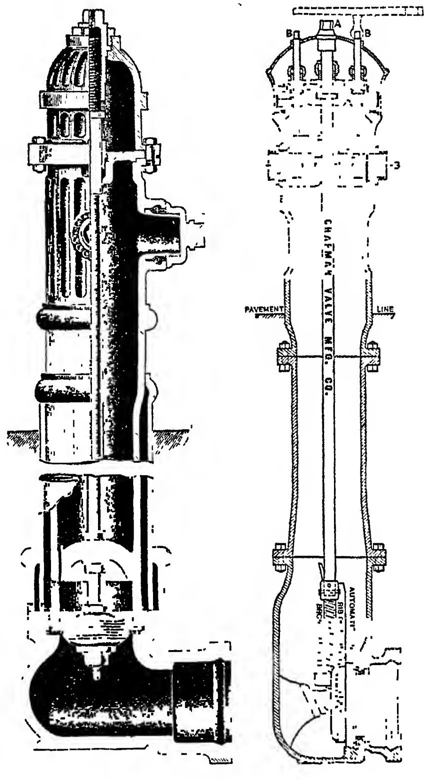


Fig. 38. Fire Hydrants.

In setting hydrants care should be taken to provide a firm base and to ram solidly back of the barrel. The hydrant branch should be covered at least as deep as the main, as this branch is essentially a dead end and is much more likely to freeze than the main itself.

96. Service Connections. Service pipes are usually from $\frac{3}{4}$ inch to 1 inch in diameter, and are made of lead, galvanized iron or tin-lined iron pipe. In making the connection between service pipe and main, the latter is tapped and a brass "corporation" cock screwed in. At the curb is usually placed another stop cock, with a suitable valve box, at which point the supply to the consumer is controlled.

Where pipes are laid in city streets, special care must be taken in backfilling and replacing the pavement. There is a wide difference of opinion as to the best method of backfilling, but probably the most certain way of getting the earth back without trouble from future settlement is by very thorough ramming of the material in a moist condition, but not wet. Such thorough ramming is difficult to secure, and it will usually be advisable to adopt the method of backfilling through a good depth of water. Hydrants are often deranged by being used for filling sprinkling carts. It is much preferable to provide water cranes for this purpose, numerous forms of which are on the market.

All constructive features pertaining to the distributing system should be carefully recorded on maps of adequate size and suitably indexed. The exact location of pipes, hydrants, and valves is of special importance. It will be convenient to have two sets of maps for this purpose—one on a small scale showing arrangement and size of piping and points of connection, and a set of large-scale maps, each one showing a comparatively small section of the system, on which the detailed information can be recorded.

OPERATION AND MAINTENANCE.

97. The maintenance of conduits and large pipe lines involves chiefly the work of cleaning and repairing. The various special structures should be frequently inspected to detect any sign of weakness, and in the case of large aqueducts the entire line should be regularly patrolled. If the water carries sediment and has a low velocity, the pipe line should be occasionally flushed by opening the blow-off valves.

Masonry conduits are likely to become coated with slime and organic growth, which will cause a large diminution of their carrying capacity, and if allowed to remain may affect the quality of the water. In such a case the aqueduct should be cleaned regularly once or twice a year, or at longer intervals, depending on the rapidity of the accumulations.

Large steel and cast-iron pipe lines will rarely need to be emptied for cleaning; but in some cases accumulations of organic growth have formed, which greatly obstructed the flow and which could not be removed by blowing off. In certain waters, particularly those

relatively soft, the interior of cast-iron pipes corrode quite rapidly as explained elsewhere. This tuberculation, as it is called, often seriously reduces the carrying capacity of the pipe. The removal of such incrustation will restore a large part of the lost capacity, and may be a much more economical method of increasing the pressure in a system than by adding new pipes.

Large pipes can be cleaned by sending workmen through them, but ordinary pipes can only be cleaned by flushing or sending through them some form of mechanical scraper which nearly fills the pipe and which is propelled by the water pressure. Very badly corroded pipes have been successfully cleaned in this way.

To remove sediment from the pipe system use is made of blow-off valves or hydrants. Dead ends may need quite frequent flushing on account of odors and bad tastes developing in the stagnant water. Large leaks in mains will quickly make themselves known, especially if a recording pressure gauge is in use. Prompt action in shutting off the supply is often necessary to prevent heavy damage. Small leaks, if occurring in clay soil, will usually be indicated by the appearance of water at the surface, but in porous soils, and especially near sewers or drains, quite large leaks may go unnoticed for years.

A serious form of corrosion which has given trouble in many cities is the electrolysis which is caused by return currents from single-trolley electric railways. In this system the return current is supposed to pass through the rails, but as these are not insulated, a portion passes through the earth to neighboring pipes or other conductors leading in the right direction. This current then flows along the pipe with more or less resistance until it reaches a neighborhood where the rails or some other conductors are of lower potential than the pipe, this being usually in the vicinity of the power station. The current then leaves the pipe, and in so doing sets up corrosive electrolytic action.

Electrolytic corrosion is in some cases so rapid that pipes are practically eaten through in three to four years, and some of the worst cases have occurred where the pressure is but $1\frac{1}{2}$ volts. The remedies for electrolysis should apparently rest entirely with the railway companies. A very important aid in preventing electrolysis is the construction of a good return conductor by means of good rail bonding and the use of adequate return wires. Then in those

districts where the pipes are of higher potential than the rails, if good, low resistance connections are made between rails and pipes, or from pipes to special return wires, the current will leave the pipes without passing into the ground and without causing trouble. Voltmeter tests between pipes and rails, at various points over the city, will determine the danger area.

Not infrequently considerable trouble arises from the freezing of service pipes which are not placed at a sufficient depth. Occasionally, also, small mains are frozen. Where the proper facilities exist the best way to thaw frozen pipes is by warming them with an electric current. For thawing service pipes a current of 200 to 300 amperes at a pressure of 50 volts is satisfactory, and will ordinarily thaw a pipe in from 20 to 30 minutes. The current can conveniently be taken from electric-light wires and reduced by a transformer.

Where the electrical method cannot be used steam may be employed, not only to warm the pipe, but to excavate through the frozen ground in a way similar to the operation of the water jet. The pipe may thus be reached at points 4 to 5 feet apart and gradually thawed out. Service pipes are often thawed by the use of a small steam pipe inserted in the service pipe through the house end, or from an opening at an excavation outside. Ground may be thawed by maintaining a fire on the surface for several hours, or more readily by the use of a gas flame projected against the soil.

Valves should be inspected occasionally to detect leakage and to ascertain if they are in working order and the boxes clean. Fire hydrants require very careful attention, especially in cold climates, as it is of the greatest importance that they be at all times available. The chief trouble with fire hydrants is from the freezing of the valves due to imperfect drainage, although a hydrant branch sometimes freezes up.

Hydrants should be carefully examined on the approach of cold weather and put in good condition. Valves should be tight and the hydrant thoroughly drained. If so located that the hydrant cannot be drained, it should be pumped out each time after being used. To ascertain if a hydrant is drained, a lead weight tied to a graduated cord can be let down through a nozzle. Hydrants should never be opened unnecessarily in cold weather, and never by others than those responsible for their condition. In very cold climates

it is found desirable after using a hydrant to oil the packing and the nut at the top with kerosene in order to prevent sticking of the valve and nut.

To thaw frozen hydrants, a small portable steam boiler is commonly employed, which is provided with a length of hose for conducting steam to the bottom of the hydrant. Hot water may also be used, and for mild cases a little salt may be effective. After thawing, the water should always be pumped out.

In the management of the pumping station the best results can only be obtained by employing thoroughly competent men. The item most susceptible of variation is the cost of coal, and every effort should be made to reduce this to the lowest practicable limit. A daily record should be kept of the weight of coal and of ashes, so that the efficiency of the service can be known at all times. Reserve machinery should be operated frequently to make sure it is in good condition and can be started when called for. This is especially important where it is depended upon for fire pressure.

Records should, of course, be kept of the amount of water pumped per day, and the pressure maintained; also of the time during which special fire pressure is furnished, and the amount of water pumped at this pressure. Recording pressure gauges are of the greatest value in maintaining the efficiency of a plant.

The maintenance of earthen reservoirs calls for little more than has already been mentioned in section 76. The cleaning of such reservoirs may need to be done frequently. It is usually accomplished by flushing out the mud through the waste pipe by means of a hose, as in the cleaning of settling basins. Standpipes and tanks may require occasional flushing or blowing out, and will need to be repainted at intervals of a few years. They should also be inspected for signs of excessive corrosion or of electrolysis, and for any indication of weakness or wear at the base. Wooden tanks need rigid and frequent inspection to ascertain the condition of the wood and of the hoops. One or two of the latter will probably need to be occasionally removed to determine this point.

98. Detection and Prevention of Waste. From the data given in section 6 it was made evident that a very large percentage of the water supplied to American cities is wasted by the consumer and lost by leakage. In many cities the consumption of

water is easily double the amount which can possibly be made use of, and in a very large proportion of them the wastage is fully one-third of the entire quantity supplied. This excessive use of water not only increases the cost of pumping unnecessarily, but adds to the expense in all parts of a waterworks system.

Unquestionably the easiest and most rational method of preventing the waste of water is by the use of meters, so that each consumer will pay for what he uses. It furnishes also the most equitable basis for charging up the cost of service, as by any other system the careful user is forced to pay for the water wasted by his careless neighbor. The use of meters is becoming much more general, and in most cities the larger consumers, at least, are now metered; but a very large part of the loss or waste is due to the small consumer, so that the full benefit of the system will not be felt until the use of meters becomes general. Usually much opposition is raised to the introduction of meters, but after they have been put into use the results are commonly such as to cause them to be greatly favored by the community. As a system of waste prevention it is always in service, and for that reason is far superior to any system of inspection. In nearly all cases the decrease in cost of supplying water after the adoption of meters much more than balances the cost of the meters.

If meters are not used, some method of inspection is highly desirable whereby the most serious cases of waste can be detected and the consumption kept within reasonable limits. The most common method is a house-to-house inspection, carried out one or more times per year for the purpose of examining the plumbing fixtures. Any leaky or imperfect fixture is ordered repaired, and the premises re-inspected shortly to make sure that the order has been complied with. Persistent refusal is followed by the shutting off of the supply.

One of the weak points of the meter system is that it fails to detect leaks in the mains or in the services beyond the meters. To localize a leak in a main, a waterphone may be used, which consists of a staff of wood or iron having at one end a diaphragm and ear piece similar to a telephone receiver. The staff is placed against the pavement over the pipe at various points, and the ear applied to the receiver, when any sound made by a leak is readily perceived.

Many different kinds of meters are on the market, most of which will give satisfactory service if properly treated, and many of them

have been thoroughly tested by years of use. No new form of meter should be adopted without thorough and long continued tests, and in all cases it is well to specify the desired requirements of a meter, and to test all new meters, in order to insure uniformly good workmanship.

The general requirements of a meter are—a fair degree of accuracy, ability to register approximately quite small rates of flow, suitable capacity for a given loss of head, durability, and low cost. All of these requirements except that of durability can readily be determined by a brief test. Some notion of the durability can also be had by a careful inspection of the parts, and by running a meter at a rapid rate for a considerable period and again determining its accuracy and sensitiveness. Maintained accuracy, accessibility, and ease of repairs are the most important qualities of a meter.

Meters should be so designed that the various parts will be easily accessible and readily replaced, and the moving parts protected from serious injury by frost. The latter object is usually accomplished by frost bottoms of cast iron, or cast-iron cases, made so as to be more easily broken than other and more costly parts of the meter.

99. Water Rates. The several services performed by a waterworks are: (1) to furnish water for private use; (2) to furnish water for public use on the streets, and for sewers, fountains, public buildings, etc.; and (3) to furnish fire protection to property. In (1) and (2) the cost of service may be considered approximately proportional to the quantity of water supplied, but in (3) it is out of all proportion to the amount of water used, for while the cost of construction is greatly affected, the total amount of water consumed is slight. The extra cost involved in furnishing adequate fire protection is due to largely increased pumping capacity, increased size of mains, reservoirs, or standpipes, cost of hydrants, and increased cost of maintenance. Estimates of careful observers place the proportion of cost chargeable to fire protection at one-third or one-half the entire cost.

The sources of revenue are the water rates and the fund received by general taxation. The former are paid by those who use the water, and more or less in proportion to the amount used. The latter are paid by assessment on all taxable property. If the revenue be so raised that each interest served be charged according to the

cost of the service, it would appear from the preceding section that the cost of furnishing water to private consumers should be paid by water rates; that the cost of supplying water for public purposes should be paid by taxation and according to the amount of water used; and that the cost of fire protection should also be met by taxation, since the individual is benefited by reason of the protection afforded to property.

The exact proportion of the revenue which should be derived from each source depends much upon local conditions, such as size of town, character of supply, etc. In many small towns the works are primarily installed for fire-protection purposes, in which case nearly all the expense should be met by taxation. It is also good policy to begin with fairly low water rates, so as to encourage the use of water, but to enable this to be done a large proportion of the expense will have to be met for a few years by taxation.

The proportion of the revenue to be derived from private consumers requires careful consideration in its adjustment. The most equitable method of apportioning the cost is by the meter system. In fixing rates under this system, allowance should be made for the fact that quite a large percentage of the water recorded at the pumping station cannot be accounted for, and rates per unit of volumes registered by the meters must be correspondingly raised.

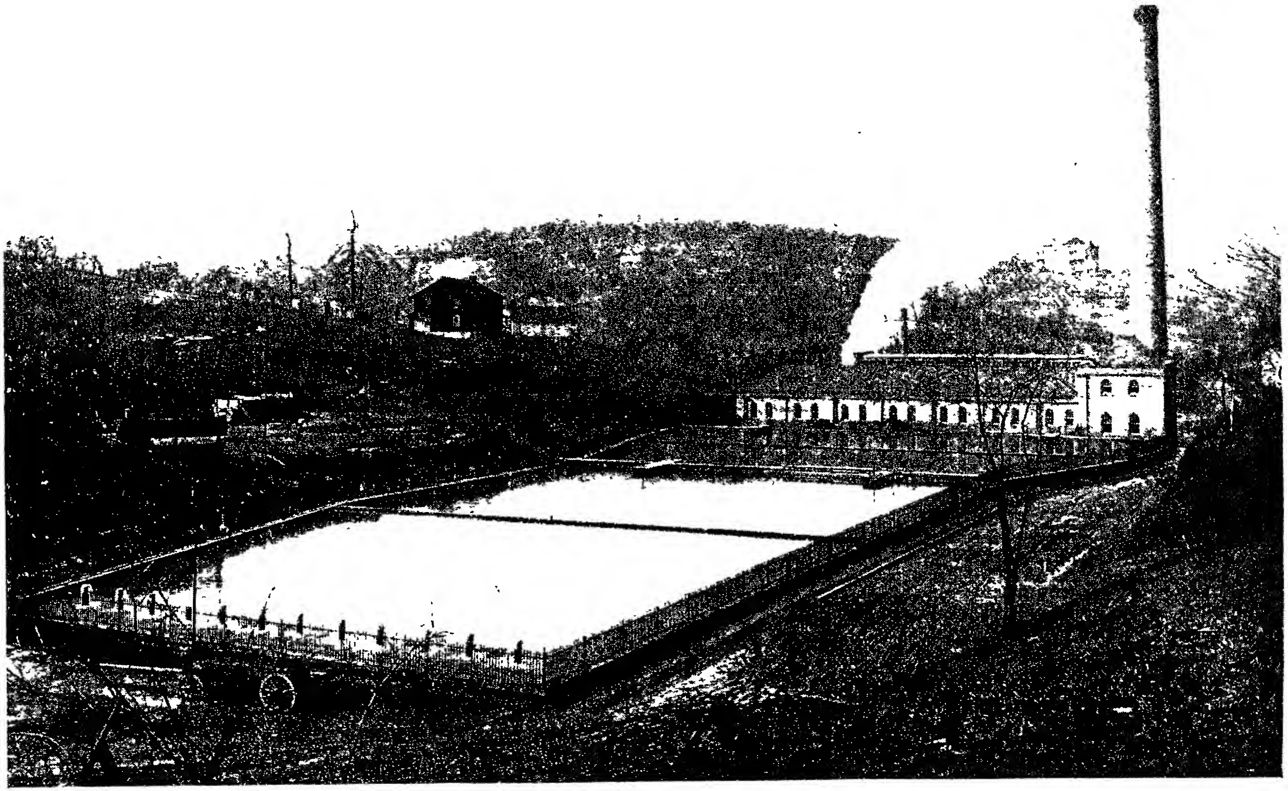
Meter rates are usually graduated, that is, a less rate is charged for large quantities than for small ones. This is partly on the ground that the cost of meter maintenance, keeping of accounts, etc., is proportionally greater for small quantities, and partly by reason of the policy of encouraging the operation of factories which contribute largely to the general prosperity of the community, and which may require large amounts of water. In establishing a graduated schedule, it should be so made that the lower rate shall apply only to the additional water used beyond the limit of the next higher rate. A good example of such a schedule is as follows:

For the first	5,000	cu. ft.	per	6 months,	20	cts.	per	100	cu. ft.
" " next	15,000	"	"	"	10	"	"	100	"
" " "	10,000	"	"	"	5	"	"	100	"
" " "	30,000	"	"	"	3	"	"	100	"
" " "	30,000	"	"	"	2	"	"	100	"
" over	90,000	"	"	"	5	"	"	100	"

A minimum charge of $2.00 per 6 months is made.

An objection to the meter system which is often advanced is that it discourages the use of sufficient water for sanitary purposes, but this is entirely obviated by making a small minimum charge, such as given above, which will be enough to allow the use of an abundance of water for sanitary purposes, and at the same time will cover the expense of meter maintenance.

Most cities meter the larger consumers, but comparatively few have yet introduced the full meter system. In such cases private houses are charged mainly by the fixture. Usually a minimum family rate is charged for kitchen use, then an additional rate for each bath tub, water closet, wash bowl, stable hose, lawn hose, etc., with often other variations depending upon the number of rooms, number of occupants of the house, etc. Little data exists as to the actual amount of water used by different fixtures, and the rates are largely arbitrary.



VIEW OF SOUTH PITTSBURGH PUMPING STATION AND FILTER PLANT

Designed and Constructed by Mr. J. N. Chester, Civil Engineer, Pittsburgh, Pennsylvania

WATER SUPPLY

PART III

PURIFICATION OF WATER*

100. Object and Methods. In the purification of public water supplies the primary object is usually to remove from the water any traces of pollution that may give rise to disease, or, in general, to remove any disease germs that may possibly infect the supply. It is often important also to remove the suspended matter where the water is turbid. Sometimes also the water contains so much dissolved mineral matter that it is desirable to remove a part of this to render the water more suitable for manufacturing as well as for domestic purposes. Thus, a very hard water is undesirable to use for boiler purposes or for culinary and laundry uses.

The various processes of purification may be divided into two general groups: those for the removal of suspended impurities; and those for the removal of dissolved impurities. Of the first class there are two general processes—sedimentation and filtration—both of which may be called natural processes.

By sedimentation water may be more or less freed of its suspended matters, including the bacteria, the efficiency of the treatment depending much upon the element of time. The process is carried out artificially in large storage reservoirs or in small special settling basins. It is often aided by the introduction of some chemical that will produce a precipitate which settles and carries down the more finely divided matter in suspension. Variations in the method of operation of settling basins and in the introduction of the chemical give rise to various modifications of the process.

Filtration is accomplished in different ways. The most common is by means of the artificial sand filter bed, either as contained in masonry basins of large size, or confined in small tanks as in the so-called mechanical or rapid filters. The chief object in all cases is

*In the preparation of Part III, the author has drawn freely from his larger work on "Public Water Supplies", Wiley and Sons, New York.

the removal of the suspended matters, and in most public supplies particular attention is paid to the removal of bacteria. The processes for the removal of dissolved impurities include the softening process, in which lime and magnesia are removed by chemical precipitation, and the process for the removal of iron in a similar manner. Such methods usually involve subsequent sedimentation or filtration for the removal of the precipitate.

Besides the foregoing general processes there should be mentioned the method of purification by distillation in which practically all impurities are removed, and the various methods of sterilization whereby the bacteria are simply killed. The latter process has come into very common use since 1910, either as the only treatment where a water is clear and not badly polluted, or as used in combination with sedimentation or filtration.

101. Tests of Efficiency of Water Purification. The object of water purification being primarily to prevent the transmission of disease, the methods of testing the efficiency of a process must be based upon a test of the water with reference to its disease-producing character. The particular diseases we are concerned with in this connection are those germ diseases in which the germs may be carried in the water supply. The two most important water-transmitted diseases are probably cholera and typhoid fever. The former is now of rare occurrence in Europe and North America, but the latter is widely prevalent. Until recent years typhoid fever prevailed to an excessive degree in many of our large cities because of the use of polluted surface-water supplies. Polluted water may also be the cause of various other intestinal troubles, but typhoid fever is the one mainly to be feared.

In analyzing a water with respect to the efficiency of a filter, a common method of measure is merely to count the total number of bacteria in a very small amount of water, and to calculate from this the average number per cubic centimeter, or per c.c., as it is usually expressed. Actual disease germs are very difficult to find, and it is usually impracticable to test a water in this way. It may be assumed, however, if a purification process removes 99 per cent of the total number of bacteria originally in the water, that the same percentage of disease germs will also be removed, so that a mere count will in most cases be of very great value. Polluted

river water is likely to contain several thousand bacteria per c.c., while this same water when purified by modern efficient processes will contain often as low as only 20 or 30 bacteria per c.c. Such a high percentage of purification is certain to be very effective, as has been proven in many cases by a great reduction in the typhoid death rate.

Another aid in bacterial examination is the determination of the presence of *bacterium coli communis*. These bacteria are an easily recognized species characteristic of sewage and other wastes of animal origin. It is highly desirable that a purification plant operate in such a manner that *b. coli* are rarely found in the purified water. In what follows, the efficiency of purification processes is often shown by the reduction in bacteria per cubic centimeter or in the absence of *b. coli*.

Each problem in water purification demands individual treatment; the best method to adopt in any case will depend upon the character of the water and the use to which it will be put, both of which elements are subject to many variations. No one process is universally applicable; furthermore, of two processes for removing the same kind of impurity, the most efficient may not in all cases be the best. A method of the highest efficiency is not always necessary, and in such cases a very substantial economy may be secured by the adoption of a system of somewhat less efficiency and of lower cost.

SEDIMENTATION AND COAGULATION

102. Value and Importance of Sedimentation. In the cases of many surface supplies, the waters contain at various times large quantities of suspended matter, either with or without more serious polluting substances, and a considerable part of the work of purification consists in the removal of these suspended matters so as to improve the physical appearance of the waters. In the case of large ponds and lakes, the period of repose is often sufficient to produce a perfectly clear water by the process of sedimentation. Artificially, however, it is not generally practicable to secure results in this manner, as the period of sedimentation must usually be limited to a few days at most.

Where a water contains little that is objectionable besides inorganic sediment, a degree of purification can often be obtained by mere sedimentation which will render the water fairly acceptable. In many instances, however, a satisfactory water cannot be obtained without subsequent filtration; but in this case the process of sedimentation constitutes a very valuable and almost indispensable prerequisite to the final treatment. For a sewage-polluted water, sedimentation alone is an inadequate treatment, as the bacteria are not eliminated in sufficient numbers to insure safety from infection. There are two general methods of sedimentation which will now be considered: plain sedimentation; and sedimentation with the addition of a coagulant.

PLAIN SEDIMENTATION

103. Action of Sedimentation. Particles of sand and clay in water have a specific gravity of about 2.6; they therefore are held in suspension only because of the currents maintained in the water. When these currents become retarded, the suspended matter is gradually deposited. The rate at which these particles will settle depends upon their size; large particles settling more rapidly than small ones for the reason that the weight of the particles increases as the cubes of their diameters, while their surface areas and resistance to settlement increase only with the square of their diameters.

To cause the deposition of the finer sediment it is necessary for the water to be brought as nearly as possible to a state of rest. In the cases of the Missouri and Mississippi river waters, and those of similar clay-carrying streams, complete clarification by simple sedimentation is impossible at certain seasons of the year, because the extremely attenuated character of the clay particles keeps them in suspension.

104. Time Required. The time required for satisfactory sedimentation is very different for different waters, and to determine this period recourse must be had to actual experiments. For some waters it requires weeks and even months to remove all the turbidity, while for others a settlement of a day or two accomplishes fairly good results. Measured by *weight* a very large part of the sediment will settle in a day or two, but the reduction in turbidity is not correspondingly great as it is the fine particles which have

the greatest influence on the appearance of the water and these remain in suspension for a longer time.

When the purpose of plain sedimentation is to prepare the water for further treatment, a high degree of clarification is not needed, it being more economical to perfect the process by other means. The best period of sedimentation will thus depend upon the character of the raw water and upon the relation of the sedimentation to the operation of the entire plant. For plain sedimentation, a period of 24 hours' subsidence is about the minimum limit adopted, although under certain conditions a still shorter period may be advisable.

Experiments show that in the cases of the Ohio and Missouri river waters, from 60 to 80 per cent of the sediment will be removed in 24 hours. The Mississippi River water at New Orleans contains very fine sediment, only about 45 per cent being removed in 24 hours, and 60 per cent in 72 hours.

105. Bacterial Efficiency. Bacteria, being extremely small in size, are very slow to settle. In fact the chief action in reducing the bacterial content by sedimentation is the action of the clay particles in carrying down the bacteria with them as they settle. The relative removal of bacteria will therefore correspond to some extent with the removal of the finer particles or sediment. Subsidence for many days will show a high degree of purification but the effect of one or two days will usually be small as evidenced by the case in the preceding paragraph.

Experiments at St. Louis gave the following results:

Time of Sedimentation (days)	Bacteria Present (per c.c.)
0	6510
1	6290
2	230
3	200

At Cincinnati about 75 per cent of the bacteria were removed in three days.

Notwithstanding the marked degrees of purification by sedimentation, this method should not be relied upon to purify a sewage-polluted water. For such a water a more efficient system should be

used, a bacterial purification of over 99 per cent being usually obtained by satisfactory processes.

SEDIMENTATION WITH COAGULATION

106. Use of Coagulants. Various chemicals, when added to water, will combine with certain substances ordinarily present, forming precipitates which are more or less gelatinous in character. These act as coagulants to collect the finely divided suspended matter into relatively large masses which are thus much more readily removed by sedimentation or filtration. Color may also frequently be removed to a large extent by this treatment. In some notable instances sedimentation, thus aided, has been found to be sufficient without further treatment. Where waters are very turbid, it will usually be more economical to allow the coarser sediment to settle before the application of a coagulant, as in this way the amount of chemical required is much reduced.

Several substances can be used as coagulants. That most commonly employed is *sulphate of alumina.* When this substance is introduced into water containing carbonates and bicarbonates of lime and magnesia, it is decomposed, the sulphuric acid forming sulphates with the lime and magnesia, while the carbonic acid is set free, and the alumina unites with water to form a bulky gelatinous hydrate which constitutes the coagulating agent. If more sulphate is used than can combine with the quantity of carbonates present, it will remain dissolved in the water, a result which it is necessary to avoid on account of the possibly injurious effect of the alum. If the water does not naturally contain a sufficient amount of alkalinity to decompose the necessary amount of coagulant, lime should previously be added to the water. Theoretically, one grain of sulphate will decompose about 8 parts per million of $CaCO_3$ or its equivalent, but, owing to the absorptive action previously mentioned, the actual reduction of alkalinity is likely to be considerably less.

Iron in various forms has also been long used as a coagulant, and is now generally employed in the form of ferrous sulphate with caustic lime. The lime is usually introduced as milk or lime. In this process, as in the alum process, the sulphuric acid unites with the lime and magnesia present, forming soluble sulphates, and the

iron forms a hydrate similar in character to the aluminum hydrate. Without the addition of caustic lime the iron would form a carbonate which would change to the hydrate but slowly. The lime unites with the free CO_2 present, thus greatly hastening the process, and at the same time precipitating part of the lime present ($CaCO_3$) in the same manner as in the lime-softening process. Very soft waters require a more exact proportioning of chemicals than waters somewhat hard, as in the latter case any excess of lime serves only to partially soften the water.

The ferric hydrate seems to be quite as efficient a coagulating agent as aluminum hydrate, and, as its cost is considerably less, the iron-and-lime process is likely to be more economical in those waters where experiments show that it can be used with success. On the other hand, sulphate of alumina appears to be of more general applicability for waters of all kinds.

Lime is another substance that may be used as a coagulant. When used in the ordinary Clark process for softening water, the effect is considerable, but still greater effects can be obtained by using lime in moderate excess. Naturally the pulverulent precipitate of lime carbonate is generally not nearly as effective as the gelatinous alumina precipitate. Experiments involving this process at Cincinnati, Ohio, showed the following average results:

	Suspended Matter (parts per million)	Bacteria Present (per c c)
River Water	273	23,800
Effluent	35	1,300
Per Cent Removed	87.2	94.5

107. Amount of Chemical Required. This depends upon the amount and character of the sediment, upon the degree of purification desired, and upon the time of settlement. It varies in practice from about three-fourths grain to 3 or 4 grains of sulphate per gallon. The proper amount can only be determined by experiment. In general the more chemical used the greater the effect, and by using a sufficient quantity and allowing enough time for sedimentation a clear water can be secured. But the question of economy will usually limit the efficiency obtained, and, where the process is but a preliminary treatment, a high degree of efficiency is not necessary.

The amount of chemical required for the Missouri River water at Kansas City, where sedimentation is the only purification method employed, is given by Kiersted as follows:

Suspended Matter after 24 Hours' Natural Subsidence (parts per million)	Sulphate of Aluminum Required for Clarification (grains per gallon)
50	0.0
100	0.5
150	1.0
200	1.5
250	1.9
300	2.4
350	2.9
400	3.4
450	3.8
500	4.3
550	4.8
600	5.3

At Cincinnati, during 1914, an average of 1.4 grains of iron and .8 grain of lime were used per gallon.

108. Time Required. The rate of sedimentation depends greatly upon the amount of coagulant employed. The settling takes place much more quickly than where no coagulant is used, so that a large part of the action will occur in a few hours. Where the process is preliminary to rapid filtration, the period allowed is usually from 2 to 6 hours. At New Orleans a period of 6 hours is provided as preliminary to filtration. At St. Louis, where the basins are operated in series, about 3 days' time is required for the water to pass through the settling basins. At Cincinnati a period of 5 or 6 days is allowed for preliminary sedimentation, and then 6 to 8 hours more after the coagulant is applied. The questions of amount of chemical needed, time of subsidence, and degree of purification desired are intimately related, and the best and most economical arrangement must be worked out for each case individually.

109. Efficiency of Sedimentation with Coagulation. Efficiency is a function of the time, amount of coagulant, and character of the sediment. The bacterial efficiency follows in a general way the efficiency with respect to the suspended matter. Where used as a preliminary process, there is usually no difficulty in securing a

sufficient degree of clarification in a few hours, either with or without preliminary natural sedimentation; the only questions being those of the amount of coagulant and the cost of operation. Where the process is final, the absolute efficiency, both with respect to suspended matter and bacteria, is of great importance.

At St. Louis, where the water passes in series through six basins, the results in 1913, were as follows:

	Suspended Solids (parts per million)	Bacteria Present (per c.c.)
River Water	1,444	57,000
Basin 1	14.0	933
Basin 2	12.1	...
Basin 3	8.4	500
Basin 4	7.1	...
Basin 5	5.8	100
Basin 6	5.6	...

In the case of the usual sedimentation of 2 to 6 hours secured in connection with rapid filtration, the reduction in bacterial content will usually range from 50 per cent to as high as 90 or 95 per cent. The latter extreme is, however, unusual. With large amounts of coagulant, such as 5 or 6 grains per gallon, very high efficiencies may be reached.

In general it may be said that the results of sedimentation with coagulation are not sufficiently good to make this a safe process to apply, without further treatment, to a sewage-polluted stream. Many waters can be satisfactorily clarified of sediment in this way, but in the case of some waters perfectly satisfactory results cannot readily be secured without filtration. The removal of color depends much upon the nature of the water. Usually from 70 to 90 per cent of the color of ordinary waters may be removed by suitable quantities of chemical, but some waters, especially those having a high color, cannot readily be decolorized in this way.

CONSTRUCTION AND OPERATION OF SETTLING BASINS

110. Methods of Operation. Settling basins are usually supplied with water by means of low-service pumps, and from the basins the water flows into an equalizing clear-water reservoir, or to a pump well, or to filters, as the case may be.

There are two general methods of operating settling basins:

The continuous-flow method; and the intermittent or fill-and-draw method. In the former, the water is allowed to flow at a very slow velocity through one or more reservoirs, during which time the settling takes place. In the latter, the water is let into a basin and allowed to remain quiescent during the period of subsidence. It is then drawn off to as low a level as efficient clarification has reached, and the basin is refilled.

In the fill-and-draw method no settlement of fine particles can commence until the operation of filling is completed, which condition materially reduces the time of subsidence. On the other hand, the water becomes more quiet than in the other process, and this operates to its advantage. Generally speaking, the continuous-flow system is the more advantageous and is the system now almost universally employed where the water is given a relatively brief period of sedimentation with the aid of a coagulant.

If the basins are operated on the continuous system, a single basin can be made to suffice, an arrangement quite suitable for a relatively clear water where sedimentation is a secondary matter, or merely a preparation for filtration. If there is much sediment, at least 2 basins are needed, in order that one may be cleaned without interrupting the supply. It is found also that generally better results can be obtained by the use of 2 or 3 basins in series than by the use of a single one of the same total capacity. With the fill-and-draw method, the number becomes a question of economical construction and operation. This will usually be from 4 to 6.

111. Form of Construction. In general, settling-basins, where large, are built similar to ordinary reservoirs, partly in excavation and partly by embankment, so as to secure the greatest economy. Earthen slopes will usually be cheaper than masonry walls, but with the fill-and-draw method the former have the disadvantage of exposing the mud at each period of emptying. They are, however, more often used. Where built for use as coagulating basins in connection with filters, they are built frequently, in the case of small plants, as part of a structural unit, being made with masonry or concrete walls and possibly floored over. The depth of basins is made so as to give the most economical construction, very shallow basins being avoided. The time of settlement is found not to be materially affected by depth.

112. Arrangement of Inlet and Outlet. The object to be attained in the continuous-flow system is the distribution of the water on entering as evenly as may be across one side or one end so that it shall enter with as little disturbance as possible; then to draw it off in a similar manner from the opposite side, and from the stratum of clearest water. As far as possible, all parts of the water should remain in the basin equal lengths of time, and all strong currents should be avoided. A common form of inlet consists in a single large pipe laid through the embankment, or a single sluice gate in a gate chamber built in the walls.

A much better distribution of the water is obtained by means of numerous inlets, or numerous branches from a single-inlet conduit, and several of the later works have been arranged in this way. The maximum uniformity of flow will usually be secured if the water is admitted near the bottom. The withdrawal of water in this system should take place from near the surface. Broad weirs formed in the wall, or made of iron troughs, are frequently used.

If a perfectly uniform movement can be secured, a single large basin will be as efficient as any other arrangement. On account, however, of the effect of wind, temperature changes, and variation in flow, there are some advantages in separating the process into parts so as to prevent the more turbid water from mixing with the less turbid. This can be done by using two or more reservoirs in series, or, less perfectly, by placing baffles or light wooden partitions in a single reservoir, or by constructing a single reservoir very long and narrow.

In the intermittent system, since the water may enter rapidly, the inlet is arranged in the simplest way, as in an ordinary reservoir. The position of the outlet is of more importance. If but a single one is used, it will need to be at the lowest point of outflow, and so will not draw from the clearest stratum except near the end of the operation. The difference in clearness at different depths after 24 hours' subsidence or more is, however, not very great. Experiments at Cincinnati showed that the upper 6 inches was considerably clearer than the water lower down, but that below this there was little change.

113. Drain Pipes. To enable the sediment to be removed, the bottom of the basin should be made slightly sloping (1 to 2 per

cent grade) toward a central drain leading to an outlet gate or to a drain pipe. The mud is removed by flushing it into the drain by means of a hose stream, supplied from a high-pressure main.

114. Preparation and Control of Coagulant. In using a coagulant, it is of the utmost importance that the introduction of the proper amount at all times be certain. This is especially true when a coagulant is depended upon in rapid filtration of sewage-polluted water where the interruption of the process would endanger the health of the community.

A common method of supplying a known quantity of coagulant is first to prepare the solution of known strength in independent mixing tanks; a duplicate set of these tanks being used. Then from one of the tanks the prepared solution is pumped or conveyed to a smaller orifice tank or dosing tank in which the liquid is maintained at a constant level, usually by applying an excess and permitting the surplus to overflow over a weir and return to the mixing tank. From this orifice tank the solution is fed through an orifice of known capacity. The head on the orifice is thus constant and the rate of flow is varied to any desired quantity through regulating the size of orifice by a hand wheel with suitable indicator.

Accurate regulation requires a knowledge of the rate of flow of the water supply as well as that of the coagulant. This is obtained from the pump counters, if pumps are used, or may be conveniently gotten by the use of Venturi meters. While being used, the contents of a mixing tank must be of uniform strength. This is accomplished by stirring with paddles, or by agitation with air, or by other mechanical means. Such agitation also aids greatly in the preparation of the solution. Lime may be used either as milk or lime, or as lime water, the latter requiring a relatively large amount of water in its preparation, but giving more uniform and reliable results.

The amount of coagulant needed is determined by frequent analyses of the water and by direct observation of results secured. In the use of alum it is very essential that there be sufficient lime present in the water to decompose all of the sulphate of aluminum used. Bronze and rubber fittings must be used in machinery for handling alum, and pipes should be of brass, bronze, or lead. Tanks and large conduits are advantageously made of reinforced concrete.

SLOW SAND FILTRATION

The first filter of which we have any record was established in 1829 for the Chelsea Water Company of London. The chief object of this filter was to remove turbidity, and in this it was a success. Its value in improving the water from a hygienic standpoint was also appreciated, although the principles underlying its action were not understood until some years later. As a consequence of the good results obtained from this filter, the filtration of all river-water supplies of London was made compulsory in 1855.

While the object of filtration at first was only to remove turbidity, it was found later that by the use of proper precautions a sand filter was also very efficient in removing bacteria and this fact enabled this method of purification to be placed on a scientific hygienic basis. Within the last twenty-five years the use of sand filters has become almost universal abroad wherever surface waters are used. In Germany it is compulsory. In the United States it is only comparatively recent that the subject has received the attention that it merits, but within the past fifteen years many efficient plants have been established in our large cities, such as Philadelphia, Pittsburgh, Cincinnati, and New Orleans.

115. Types of Filters. Sand filters are of two general types: the slow filter; and the rapid filter. The former is operated at a rate of from 2,000,000 to 6,000,000 gallons per acre per day, while the latter is generally operated at a rate of from 100,000,000 to 125,000,000 gallons per acre per day. These very great differences in rate of filtration necessitate important differences in construction and methods of operation in order to secure satisfactory and economical results, but the rate of filtration is the essential point of difference between the two types.

In the slow sand filter, the sand bed is constructed in large water-tight reservoirs, either open or covered, each having usually an area of from one-half to one and one-half acres. On the bottom of the reservoir is first laid a system of drains, then above this are placed successive layers of broken stone and gravel of decreasing size, and finally the bed of from 2 to 5 feet of sand which forms the true filter. The water flows by gravity, or is pumped, upon the filter, passes through the underdrains to a collecting well, and thence to the consumer. As the water filters through the sand, the friction

causes some loss of head, which gradually increases as the filter becomes clogged with foreign matter. The rate of filtration is, however, maintained nearly uniform by suitable regulating devices which vary the head according to the resistance. When the working head has reached a certain fixed limit of a few feet, the water is shut off, the filter is drained, and the surface is cleaned by removing a thin layer of clogged sand. The operation is then resumed. Before the thickness of the sand layer becomes too greatly reduced, clean sand is added sufficient to restore the filter to its original depth.

The chief features to consider in the slow filter are the proper construction of sand bed and drains, the rate of filtration and its regulation, the loss of head, cleaning of beds, washing of sand, and the control of the operation from bacteriological tests.

The rapid filter differs from the slow filter in many of its details. It is built in much smaller units, and the drainage system and operating devices are widely different. Furthermore, in its operation it is dependent upon the use of a coagulant for efficient results. Further discussion of this type of filter is given in the next chapter.

EFFICIENCY OF FILTRATION

116. Theory of Filter Action. When working under favorable conditions, a sand filter will remove the suspended matter and very nearly all the bacteria originally present in the water. Even the color of a peaty water may be somewhat decreased but that portion of the color due to matter in solution is not readily removed by filtration. Dissolved impurities will in general be little affected by filtration.

From the standpoint of health the efficiency of a filter in removing bacteria is the most important question. When a filter is satisfactory in this respect, it will also be found to be efficient in removing the fine sediment; and on the other hand, when a filter is not producing a clear effluent, it will usually be found that it is also not bacterially efficient.

The action of a filter is partly mechanical in straining out the larger particles of sediment, and in promoting the settlement of the finer particles in the pore spaces of the sand, but the removal of the bacteria cannot be wholly explained in this way. The efficiency of a filter in removing the bacteria, and the finer particles of sedi-

ment, is very largely due to a sort of organic slime or growth (largely bacterial) which gradually forms in a filter to a depth of several inches, but chiefly quite near the top. This slime or growth appears to be a very effective agent in securing high bacterial efficiency. Where a coagulant is used in connection with a rapid filter, the precipitate formed furnishes a good substitute for the organic material in the slow filter.

A filter is, therefore, something more than a mere mechanical strainer, inasmuch as its efficiency rests largely upon biological causes. The sand itself acts as a mechanical support for these gelatinous films, holding them intact; for this reason a certain depth of sand is necessary to steady the action of the filter and prevent disturbance of this organic slime.

117. Bacterial Efficiency. Where a slow sand filter is doing satisfactory work, the number of bacteria found in the effluent is on the average small, either when expressed absolutely or compared with the number originally present in the unfiltered water. A good deal of variation, however, exists even in the same supply, as is evident when a continuous study is made for considerable periods of time.

A common standard for the bacterial content of the effluent is a maximum of 100 bacteria per c.c., but the significance of such a number depends much upon the character of the raw water. The proportion of bacteria removed is also significant and usually varies from 98 per cent to very nearly 100 per cent. In the case of a sewage-polluted water, the presence or absence of the characteristic *bacterium coli communis* in the effluent is another and very important measure of filter efficiency.

In the case of filters using quiescent waters as sources of supply, where the number of bacteria in the applied water is low, the percentage of bacterial efficiency may be relatively low and still result in a very pure effluent. In such a case the number in the raw water may be only a few hundred and that in the effluent less than 20 or 30, which would be considered a good result, while the percentage efficiency might not exceed 95 or 96. On the other hand, where the source of supply is from running streams, the bacterial content of the raw water may be quite high and as many as 30,000 or 50,000 per c.c., in which case the effluent may contain as high as 100 under

TABLE XIV
Bacteria in Water

Season	Raw Water Bacteria (per c c)	Settled Water		Filtered Water		Removal by Settlement and Filtration Combined (per cent)
		Bacteria (per c.c)	Removal (per cent)	Bacteria (per c.c)	Removal (per cent)	
Winter	16600	6300	62	149	97.6	99.1
Spring	4150	980	76	29	97.0	99.3
Summer	4100	160	96	18	88.8	99.6
Fall	1960	270	86	22	91.8	98.9
Year	6760	1940	71	54	97.2	99.2

very efficient conditions. The percentage removal in that case would be very high. The efficiency of filtration is also much affected by variation in working conditions, as by a fluctuation in rate or by scraping the surface of the filter. Formation of ice on uncovered filters is also detrimental.

The average results for the filter plant at Washington, D. C., for the year 1909–1910 were as given in Table XIV. It is seen from Table XIV that the percentage removal is very high where the number of bacteria in the raw material is high.

118. Death Rate as a Measure of Efficiency. While the common method of measuring the efficiency of any filter is to measure it by the bacteria appearing in the effluent, either expressed absolutely or in terms of percentage removed, still, after all, the effect on the death rate or the case rate of water-borne diseases is the crucial test of efficiency. Where statistics are comparable, they invariably show a diminution in death rate that is sometimes so marked as to be astonishing.

Table XV shows the effect of the introduction of filters upon the death rate from typhoid fever in a number of cities which began the use of filtered water between 1905 and 1909.

119. Rate of Filtration. In the design of a filter plant the first question to be settled is the rate of filtration which shall be adopted. The higher the rate the less the area required and hence the less will be the first cost; but the cost of operation is not greatly affected by the rate. In general high rates of filtration will give less efficiency than low rates, but until the rate exceeds a certain amount the difference in efficiency is small.

TABLE XV

Typhoid Fever Death Rate Per 100,000 Population

City	Mortality	
	Average for Six Years 1900–1905	Average for Years 1909–1910
Cincinnati	54	10
Columbus	61	15
Philadelphia	47	20
Pittsburgh	132	13

Rates of filtration in this country usually are stated in terms of gallons per acre per day or per hour. The experience of European works has resulted in the adoption of a rate, for most places, of between 2 and 3 million gallons per acre per day, but in this country somewhat higher rates have been favored. Probably 3 or 4 million gallons is as high as it would in general be advisable to go in the design of a new plant. If subsequent operation shows that a higher rate can be adopted with efficiency and economy, the fact can be taken advantage of as the demand for water increases. It should not be overlooked that there may be cases, where, with a moderately polluted water, all the practical benefits of filtration can be secured at rates much higher than are usually employed. Each case demands independent consideration in order that the best and most economical solution may be arrived at. Sudden changes of rate are apt to produce disturbances in the filter and to give a reduced efficiency. In practice, absolute uniformity of operation is unnecessary, but sudden changes in rate should be avoided, especially any large increase above the normal.

DETAILS OF CONSTRUCTION AND OPERATION OF SLOW FILTERS

120. Size of Filters. The standard rate of filtration having been determined, the required net working capacity will be equal to the maximum rate of delivery divided by the assumed rate of filtration. To economize area and to avoid rapid changes in rate, a clear-water reservoir should be provided. The best size for this will depend on local conditions, but it will usually be desirable to

have it of sufficient capacity to equalize the demand throughout the day. It will then be necessary to vary the rate of filtration only to accord with the daily variations in consumption. In "Variations in Consumption", Water Supply, Part I, section 6, it was shown that the maximum daily rate of consumption is likely to be about 150 per cent of the average, and with a clear-water reservoir of the capacity mentioned above, the filters must be designed to deliver at this maximum daily rate.

In addition to the area as above found, a reserve area for cleaning must be provided. For small works this will be one bed; for works containing several beds it will be necessary to allow one bed extra in every 5 or 10 beds, depending on the frequency of scraping and the time required for putting a filter into operation after cleaning. The proper size of beds is chiefly a question of economical construction. The larger the beds the less the cost per acre, but the greater will be the area out of service in the one or more reserve beds. Ordinarily the size for a considerable number of beds is from 1 to 1.5 acres for open beds, and from .4 to .8 of an acre for covered beds. For small total areas of .5 to 1 acre, three beds would ordinarily be used, and for still smaller areas two beds. The economical number can in any case be determined by comparative estimates, but under ordinary conditions the number should be about as follows:

TOTAL AREA (acres)	BEDS
1	3 or 4
3	5 or 6
6	7 to 10

121. Filter Beds. Filter beds are usually rectangular in form and arranged side by side in one or two rows according to the number. The shape of the area available often determines this point, but otherwise a convenient arrangement is to place them in two rows with a space between for sand washing, etc., and to have valve chambers facing this central passage-way, as illustrated by the Albany plant, Fig. 39. A single row would be more economical of masonry but would require more piping. A large number of basins may be divided into groups and arranged in the above manner.

In general construction a filter basin is built in a way similar to small distributing reservoirs (see Water Supply, Part II, section 76).

Earth embankments for the sides are cheaper than masonry walls, but require more ground. If the filters are covered, masonry walls

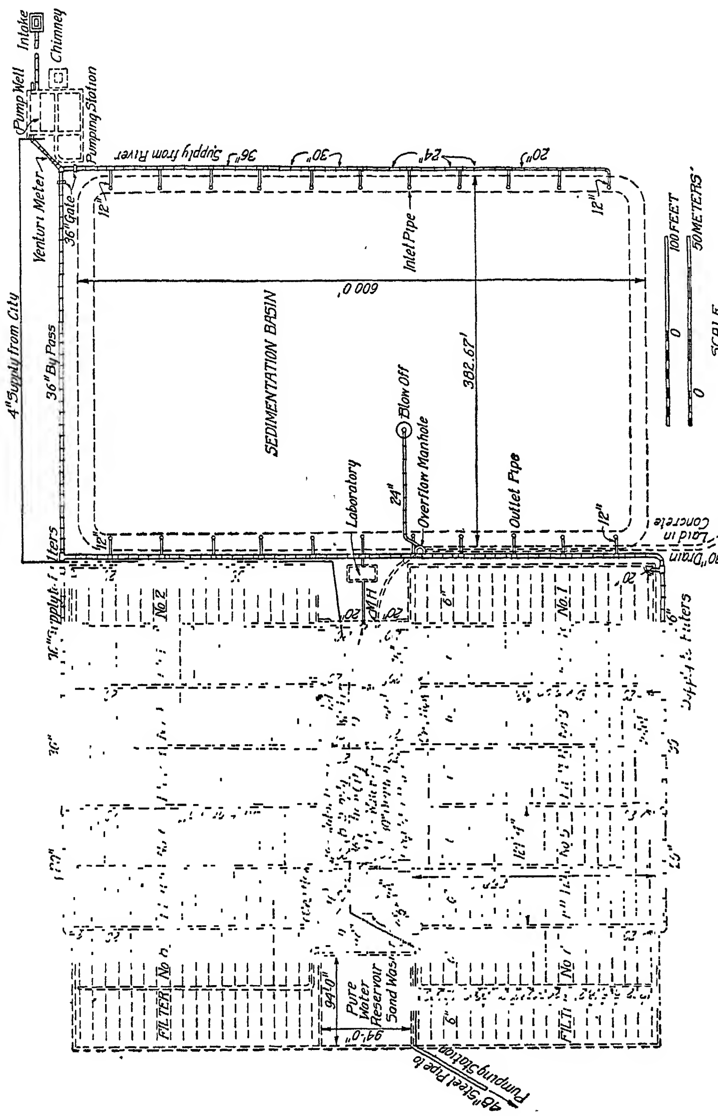


Fig. 39. Diagram of Albany Filter Beds and Sedimentation Basin
From "Transactions, Am. Soc. C. E.", Volume XLIII

are usually employed. Particular care must be taken to render the basin water-tight both on the bottom and at the sides. Cracks in

division walls are likely to admit unfiltered water to the underdrains and should be guarded against especially.

The depth of open filters is made only sufficient to contain the necessary depth of filtering materials and water, as explained subsequently, and still have a margin of 2 or 3 feet from the water surface to the top of the embankment. This will give a total depth of 9 or 10 feet. In closed filters the distance from top of sand to cover must be sufficient to give head room for workmen when cleaning the filter, a distance of about 6 feet.

122. Covers. Covers for filters are constructed of the same general form and arrangement as described in section 78 of Water Supply, Part II. Masonry or concrete vaulting is usually employed, although wood has been used; but the latter does not afford as good a protection from freezing or from summer heat. Admission for workmen is provided by a gangway leading from an opening at a point where the vaulting is raised. Walls and piers should be built with small offsets near the bottom in order to insure good filtration at that point. A covered filter is illustrated in Fig. 33.

The principal reason for covering filters is to avoid the difficulties connected with the operation of open filters in winter. To clean filters when covered with ice is a troublesome and expensive operation, requiring the removal of the ice or the use of special methods giving only inferior results. If the filters are drained for cleaning, trouble also arises from the freezing of the sand. The cleaning of beds is thus not likely to be done as promptly as desirable, and the result of winter operation will be a decreased effective area and a lowered efficiency. Whether covers should be used depends upon the extent to which ice will form, the frequency of the occurrence of thaws which will enable a filter to be properly cleaned, and the length of time between cleanings as determined by the character of the water.

123. Filter Sand. Experiments show that very fine sand is considerably more efficient in removing bacteria than ordinary or coarse sand, but within the ordinary limits of size there is but little difference in efficiency. The finer sands, however, cause a steadier action and prevent disturbances due to scraping; they also cause a greater loss of head in the filter, and so make the action more uniform over the filter area. On the other hand, fine sand becomes clogged

sooner than coarse and involves therefore more expense in cleaning.

It is desirable that a sand be fairly uniform in grain. If the particles vary greatly in size, it will be difficult to wash, and in fact will have quantities of the finer particles removed in the process, thus increasing the effective size. It is especially important that the sand should be of the same grade in all parts of the same filter in order that the frictional resistance, and therefore the rate of filtration, shall be uniform. In designing a filter it should be noted that the sand forms the filtering medium; the gravel serves simply to collect the filtered water with little resistance to flow. There is no object in having the main body of sand of different sizes.

The depth of sand must be sufficient to form an effective filter and, besides, to allow of several scrapings without renewing the sand. The effect of deep beds is similar to that of fine sand in steadying the action of a filter, and it has been clearly shown that the operation of beds 4 to 5 feet thick is not so much affected as that of beds 1 to 2 feet thick by such disturbances as variation in rate, scraping of beds, etc. A depth of 3 feet is about right, with one foot allowed for scraping.

The depth of water on the filter should be sufficient to enable the desired maximum head to be used without reducing the pressure in the filter below atmospheric; and as the resistance is nearly all at the surface of the sand, the depth must be about equal to the maximum head to be used in forcing the water through the filter. The depth must also be greater than the thickest ice likely to form. Beyond these limiting depths any increase serves only to increase the expense of construction. About 4 feet is a common value.

124. Drainage Systems. To collect the filtered water, a system of under drains is necessary. The important points to be considered in its design are durability and freedom from derangement, and that the loss of head therein shall be small. The system of drains usually consists of a large central drain running the length of the filter, and branch drains at right angles thereto placed at regular intervals, usually of 8 to 12 feet. The central drain may be either of large vitrified pipe, as in Fig. 40, or of masonry. The branch drains are usually of 4- to 8-inch round or special tile, laid with open joints.

To conduct the water to the lateral drains, coarse gravel an inch

or two in diameter is filled about the drains and spread in a layer of 6 inches or more in depth evenly over the floor of the filter, or, if the bottom of the filter is irregular, it may be arranged as shown in Fig. 40. Above this coarse gravel are then placed three or four layers of finer gravel, each successive layer being finer in size, but not so fine as to settle into the previously laid layer. The last layer is made fine enough to support the sand. The thickness of these layers need

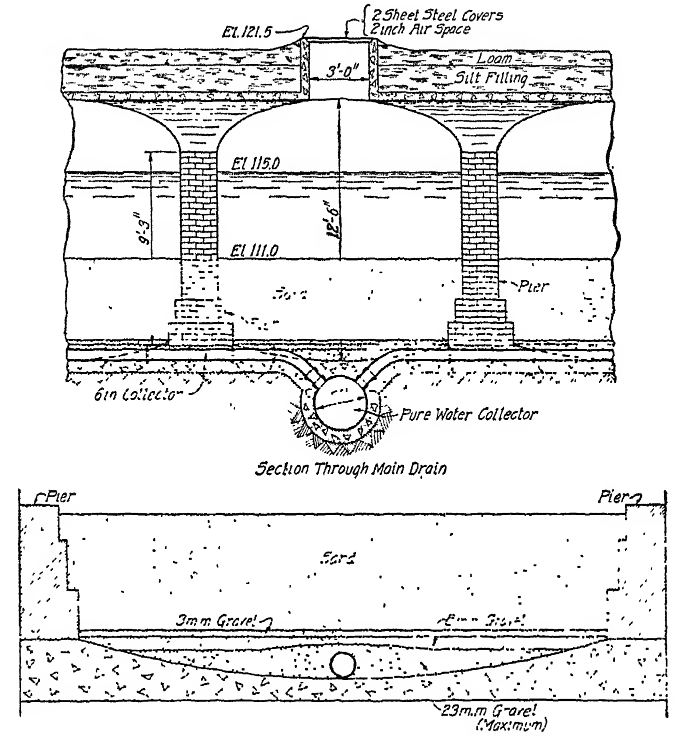


Fig. 40. Sections Showing Details of Drains for the Albany, New York, Filter Beds

be only 2 or 3 inches if carefully laid, or just sufficient to insure that the next layer below is well covered.

The gravel used should be carefully screened and, if dirty, washed. It is readily sized by revolving or fixed screens, using for this purpose three or four different sizes. The smallest should have about a $\frac{3}{16}$-inch mesh, and each larger size about double the size of mesh of the next preceding. Sizes of $\frac{3}{16}$-, $\frac{3}{8}$-, 1-, and 3-inch mesh are commonly used in grading the gravel.

125. Inlet and Outlet. Water is admitted to the filter through a single branch main at about the level of the surface of the sand. The flow is usually controlled by a balanced valve operated by a float, so as to maintain the water in the filter at a constant level. A gate valve is provided in addition, to enable the water to be completely shut off at any time. Fig. 41 illustrates the balanced float valve and details at one of the large Philadelphia plants. To avoid disturbing the sand as much as possible the water should flow upon the bed at a low velocity, and a common arrangement is to provide a broad weir, as shown in Fig. 41, over which the water passes.

In place of providing a regulating valve for each filter, the influent pipes may all lead from a central regulating well in which the water level is maintained constant. Such an arrangement is suited to a compact group of small filters.

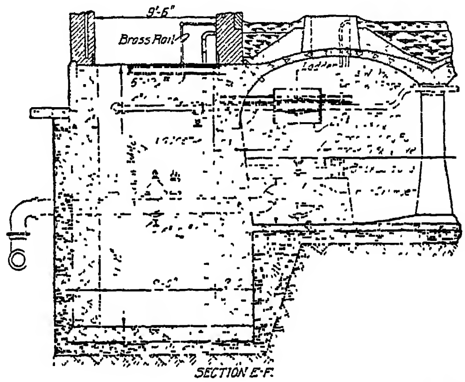


Fig. 41. Details of Inlet Regulator Used at Philadelphia Filtering Plant. Section Taken Through *EF* of Fig. 42
Courtesy of Engineering Record

If the water level on the filter is kept constant, the rate of filtration must be regulated, when the filter becomes clogged, by lowering the water level or reducing the pressure at the outlet. In the older filters no arrangement was provided for regulating each filter independently, but each was connected to the clear-water well by a short pipe fitted with an ordinary valve. The head on all filters was consequently always the same, except as it might be controlled by throttling at the valves. The effect of unequal heads on the rate of filtration, where some of the filters might be freshly cleaned and others badly clogged, can readily be imagined. Independence of action, especially as respects maximum rate, is greatly to be desired and is now the general practice.

The regulation of head requires, *first,* some form of measuring device, such as a weir, orifice, or Venturi meter by which the rate of

filtration can be ascertained at any time by floats and indicators; and, *second,* the controlling of the rate of flow either by hand or automatically. Floats are also required to show the level on the filter and the head in the main drain, the difference of which is the working head on the filter. The apparatus for regulation is placed in one or more chambers with which the main drain of the filter connects.

Automatic regulators for delivering water at a constant rate are in use in a number of places. They usually consist of a weir in the form of a telescopic tube which is supported by means of a float in the chamber connecting with the underdrain. By adjusting the

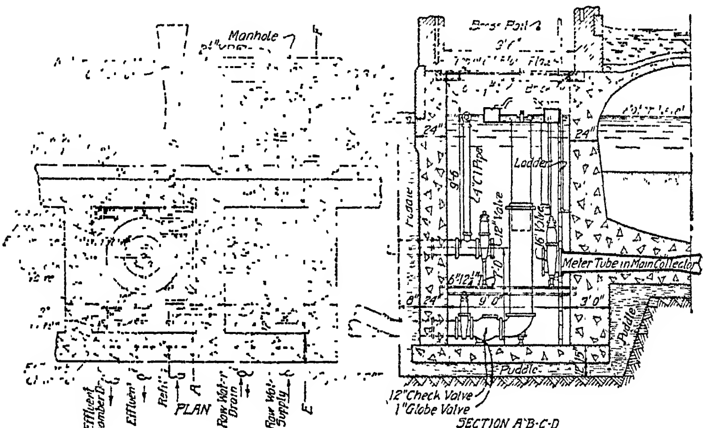


Fig. 42. Plan and Section of Automatic Regulator Used in Philadelphia Filtering Plant
Courtesy of Engineering Record

float, the edge of the weir can be maintained at any desired distance below the water surface. A weir of this general type is illustrated in Fig. 42. The rate of discharge is varied by changing the relative height of float and weir.

Besides the inlet and outlet pipes, a drain pipe must be provided through which the water may be drawn off. This is usually connected with the chamber into which the main drain opens, as shown in Fig. 42. An overflow pipe is also necessary to provide against any failure on the part of the inlet regulator. This connects with the drain pipe.

Arrangements should be made for wasting the filtered water in case it should be necessary, also for drawing off the water from above

TABLE XVI

Frictional Head in Drains per 100 Feet of Drain
(feet)

Discharge (gal. per day)	Velocity (ft. per sec.)	Drain Diameter									
		4″	6″	8″	10″	12″	15″	18″	20″	24″	30″
700 x (diam.)^2	.2	.012	.006	.004							
1400 x (diam.)^2	.4	.050	.025	.016	.011	.009	.006	.005	.004	.003	.002
2100 x (diam.)^2	.6	.113	.057	.036	.025	.019	.014	.011	.009	.007	.005
2800 x (diam.)^2	.8	.202	.101	.064	.045	.035	.025	.019	.016	.012	.009
3500 x (diam.)^2	1 0	.315	.158	.100	.070	.054	.039	.030	.025	.019	.014

a filter down close to the sand layer in order to save time in emptying; and facilities should be provided for sampling water from various points in the system. By-passes should be provided to enable either settling basin or filters to be cut out if necessity arises. For furnishing water for sand-washing and various purposes, connection must be made with high-pressure mains.

126. Loss of Head. The total loss of head in the filter is equal to the loss of head in the sand plus that in the underdrains. That in the sand is uniform throughout the filter, but in the underdrains it varies from zero near the outlet to a maximum for the most remote point. The rate of filtration will be proportional to the total head and therefore will vary in different parts of the bed. The loss of head in the drains should be kept so low that with a clean filter the variation in the rate of filtration in different parts of the bed will not be excessive. A variation of 20 to 25 per cent would not be a serious matter, as the excess above the average would then be only 10 or 12 per cent.

With a rate of filtration of 3 million gallons per acre per day, the lateral drains will need to be of such size as to keep the velocity of flow in them down to .2 or .3 of a foot per second and in the main drain a velocity of .6 to .8 of a foot per second. The necessary size of drain and the loss of head per 100 feet due to friction can be determined by Table XVI. As a filter becomes clogged the head necessary to cause filtration at the assumed rate increases. By increasing the head to a high figure, the filter can be operated longer without scraping and so a saving may be effected. On the other hand, high losses of head require more pumping and greater depth

of filter and have a detrimental effect in compacting the sand. A maximum loss of head of 4 or 5 feet is good practice.

127. Cleaning Filters. When a filter has become clogged and has reached its highest allowable loss of head, it is drained and then cleaned by removing by means of broad thin shovels a layer of clogged sand from $\frac{1}{2}$ to 1 inch in thickness. The surface is then smoothed with a rake. The sand is removed from the filter by means of wheelbarrows or as now more generally done, by portable ejectors to sand washers, where it is cleaned and stored or returned to another bed. After scraping, the filter is filled, preferably from below, with filtered water until covered 2 or 3 inches deep; then raw water is run on to the usual depth, and the filter again started. At intervals of a year or so, and before the layer of sand has been reduced below the desirable minimum thickness of about 12 inches, the bed is restored to its original depth by adding clean sand. After cleaning and filling, the filter should be started slowly and gradually.

The period of service between cleanings depends upon the character of the water, upon the fineness of the sand, and upon the maximum allowable loss of head. It is directly affected by the rate, a rapidly working filter becoming clogged proportionally sooner. In practice it varies from a few days, if the conditions are especially bad, to five or six weeks or more where the conditions are good. The amount of water filtered between cleanings ordinarily ranges from 40 to 80 million gallons per acre.

In many cases there is a considerable decrease in the efficiency of a filter for some time after scraping, and in some works it is the practice to waste the effluent for one or more days at this time. At other places it has been found sufficient to begin the operation very slowly after scraping. Filling a filter slowly from below was found to give much better results than filling from above. A good effect was also observed when the water was permitted to stand a few hours on the filter before starting the operation. If these precautions are followed, there is likely to be little need of wasting the effluent, but the necessity for this in any particular plant can be readily determined by experience. When the sand is renewed the necessity of wasting the effluent is much greater.

128. Sand Washing. Various methods have been employed for washing dirty sand, but the one now commonly used is known

as the "ejector" washer. It is a device which operates on somewhat the same principal as the steam injector, and is used both for

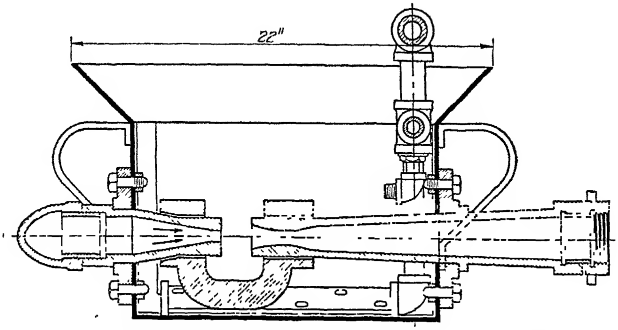


Fig. 43. Section of Portable Ejector Used in Washington, D. C., Filtering Plant

cleaning the sand and for mixing it with water so it can be elevated and transported through pipes by means of water under pressure.

When the ejector system is used, the filter plant is fitted up with high-pressure water mains, a 3- or 4-inch branch running to each filter. In removing the sand from a bed a portable ejector is connected up with the high-pressure pipe line by means of a short line of hose. The sand is then shoveled into the ejector which forces it through another line of pipes to the washer. There it is washed by other sets of ejectors and forced again through pipes to storage or back at once to one of the filters. Very little manual labor is required and the sand is handled very economically.

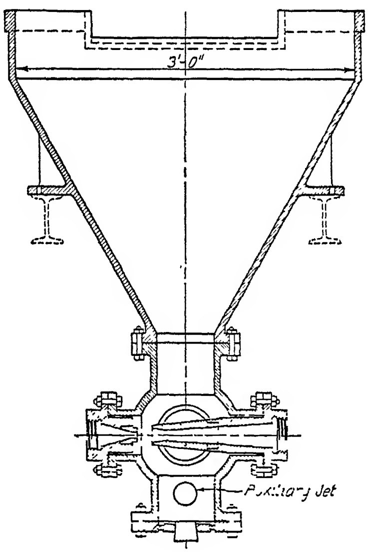


Fig. 44. Section of Ejector Washer Used in Filtering Plant at Washington, D. C.

Fig. 43 shows the portable ejector used in the Washington plant. The sand is shoveled into the steel box, is there lifted and made liquid by water forced through the perforated pipes near the bottom of the box, and is then carried away by the action of the

ejector jet. The mixed sand and water is carried through a 4-inch pipe to the washer. Fig. 44 shows the ejector used for washing purposes. The sand, containing a large proportion of water, is discharged into the hopper from above, the water overflowing the edge and carrying the dirt away with it. The clean sand settles and is forced out through the ejector at the bottom. As the ejector tends to carry out more water than it supplies, some of the dirty water from above would be carried along with the sand if no additional water were supplied. To avoid this an auxiliary supply is introduced near the bottom sufficient in amount to prevent a downward current. By this means a single hopper will effect good results although two hoppers are provided which may be operated in series.

129. Control of Filter Operations. The most accurate way in which to gage the operation of filter plants is to subject the water to a bacterial examination. This should be made at frequent intervals so as to note any possible changes in quality. The experience with European filter systems has shown that an impairment in quality has not infrequently been detected in time to prevent outbreaks of disease. In the larger filter plants, a bacteriological laboratory should be installed, and daily tests of the effluent made. The filter beds should be arranged so that the effluent from each can be tested separately, and provision made so that the filtered water can be rejected from any one filter if not up to standard. The careful control of the operations is a matter of great importance. In testing filters as to their efficiency, samples should be collected at periods when the effluent is likely to be the least favorable, as during frost periods, heavy rains, and periods of greatest consumption.

PRELIMINARY TREATMENT OF WATER FOR SLOW SAND FILTRATION

Nearly all waters contain at times suspended matter to such an amount, or of such a character as to render desirable a preliminary treatment for the removal of a portion of this sediment. This may be simply a question of the most economical method of treatment, or the water may be of such character as to render such preliminary treatment necessary for satisfactory results. Large quantities of clay or silt clog up a filter quickly, and if the sediment is very fine it penetrates deeply into the filter and may make the effluent turbid.

Preliminary treatments may consist of simple sedimentation, sedimentation with coagulation, or preliminary rapid filtration with or without coagulation and sedimentation.

130. Simple Sedimentation. In the filtration of river waters it will nearly always be economical to provide at least a few hours' preliminary sedimentation (see section 103). While river waters are most subject to great turbidity, supplies derived from lakes may also give trouble from this cause.

131. Sedimentation with Coagulation. The elaborate experiments at Cincinnati, Louisville, and New Orleans, and the accumulated experience in treating the water of the streams in the Mississippi Valley, have shown that filtration preceded only by plain sedimentation is inadequate to give satisfactory results with this water. Ordinarily from 30 to 50 parts of suspended matter per million can economically be taken care of by the filters, although it is not the amount but rather the nature that determines whether a good effluent can be secured. Where this is not possible a coagulant should be employed in connection with settling basins as already described. The construction and operation of the filters are the same. Experience with the slow sand filter plant at Washington, D. C., has also shown that perfectly clear water cannot always be secured even with the long period of sedimentation there obtained, and that some form of preliminary treatment with a coagulant appears to be necessary.

132. Preliminary Filtration. In purifying badly polluted waters, and especially those of high turbidity, some form of rapid filter may often be adopted to advantage for preliminary treatment; slow sand filters being employed for the final process. In other cases a preliminary filter may be used for reasons of economy, the increased rate thus permitted in the main filters effecting a greater saving than the cost of the preliminary treatment.

At Albany, rapid sand filters have been adopted for preliminary treatment. A rate of about 70,000,000 gallons per acre per day is used in these filters, with 12 hours' plain sedimentation. This allows the original, slow sand filter plant to be operated at a rate as high as 6,000,000 gallons per acre per day with good results.

At Philadelphia preliminary filters are used at some of the plants. These filters consist at one plant of the usual form of rapid sand

filter operated at a rate of about 80,000,000 gallons per acre per day, while at another plant they consist of Maignen "scrubbers" described in section 149. The latter are also used at South Bethlehem, Pennsylvania, where the main filters are operated at a rate of 7,000,000 gallons per acre per day.

Double, slow sand filtration is in use in a number of European works, notably at Bremen, Germany; Shiedam, Holland; and Zurich, Switzerland. Two sets of sand filters are used, operated in about the same manner, although their rates of filtration are usually different. A somewhat higher rate than normal is used in these filters.

COST OF SLOW SAND FILTER PLANTS

133. First Cost. The cost of sand filters depends greatly upon local conditions as influencing cost of excavation, cost of sand, etc. Large beds and extensive works will cost less per unit area than smaller ones, other things being equal. As a rough figure, the cost of covered filters of about one-half to one acre in area will be in the neighborhood of from $50,000 to $75,000 per acre, exclusive of land. To the cost of filters will have to be added the cost of clear-water reservoir and sedimentation basins, usually amounting to from $3,000 to $10,000 per million-gallon capacity.

134. Operating Cost. The principal operating cost items are the scraping of the filters, and the cleaning and renewal of the sand. With modern methods of sand handling, this cost amounts to only 30 to 60 cents per million gallons filtered. The entire cost of operation of the Albany filter plant in 1910 was as follows:

Division	Cost per Million Gallons Filtered
Pumping station	$2.81
Preliminary filters	.69
Slow sand filters	1.63
Laboratory	.65
Superintendence	.21
Total	$5.99

At Washington, D. C., where the filters need cleaning much less frequently, the cost of sand handling is only 30 cents per million gallons, and the total operating expense, including pumping, laboratory, and superintendence, is about $3.80 per million gallons.

RAPID SAND FILTRATION

135. General Description of Rapid Sand Filter. This type of filter, also called the "mechanical filter" and the "American filter", is a form of filter designed to accomplish results in the way of purification comparable with those obtained by the slow sand filter already discussed, but with a much smaller sand area. It is similar to the slow sand filter in that the filtering material consists of a bed of 3 or 4 feet of sand or crushed quartz, but in other respects the construction and operation are widely different. The essential points of difference are: the very rapid rate of filtration (100 to 125 million gallons per acre per day), the use of a coagulant to aid in filtration, and the manner of washing the sand bed. These peculiarities lead to noteworthy differences in construction. The units are relatively small in area; the coagulating basin together with adequate means for mixing and regulating the coagulant becomes an essential part of the plant; and the washing of the sand, which, in this type, must be done every few hours, requires the use of special devices of a more or less elaborate character. In the operation of a rapid-filter plant, the frequent attention required of each unit renders the question of compact and convenient arrangement of piping and operating valves of much importance. At the same time the small size of the unit enables this to be readily done, and a part of all the plant to be placed under roof. The washing of the sand beds is accomplished by a reverse flow of water, assisted, usually, by agitation of the sand bed by means of mechanical rakes or compressed air. The details of this part of the process constitute the chief differences between the various types of rapid filters.

The development of the rapid filter arose from the effort to settle and clarify very turbid water by the use of a coagulant, followed by rapid filtration. When bacterial purification became of greater importance the rapid filter was looked upon with much suspicion, owing to the extremely high rate of filtration used as compared to the rate employed in the better-known slow sand filter. Results of daily operation in practice, and of many special experiments have shown, however, that with proper supervision the rapid filter will give essentially the same results as the slow filter, and that in some waters the results are better than can be obtained by the slow filter without the use of a coagulant. This condition has

led to the quite general use, in the United States, of the rapid filter whenever it is the better adapted to local conditions.

A great number of devices relating to rapid filters have been

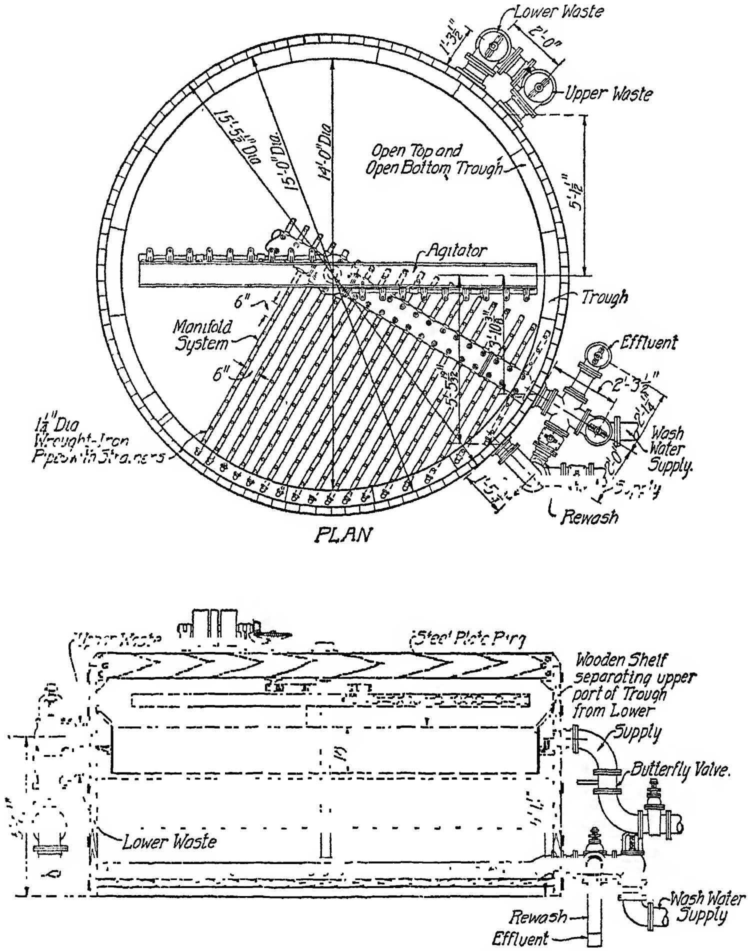


Fig. 45. Plan and Sectional Elevation of a Warren Filter at Chester, Pennsylvania
Courtesy of Engineering Record

patented, the most important perhaps being the various screen and washing arrangements as described in later sections.

136. Types of Construction. The usual form of rapid filter, as constructed and sold by the proprietary companies, consists of units made up of circular wooden or steel tanks. These contain

the sand, supported on suitable strainers, and each is equipped with piping arrangements for washing and means for agitating the sand. In one of the most common designs formerly employed each tank was divided by a horizontal partition, the lower portion acting as a coagulating chamber. The coagulating basins are now usually built separate from the filters so as to provide larger settling capacity. The form of construction here described is illustrated in Fig. 45, which shows one of the filters installed at Chester, Pennsylvania, in 1903. The filter unit consists of a cypress tank 15 feet in diameter containing a sand bed of $2\frac{1}{2}$ feet thick. This is supported on a layer of gravel, near the bottom of which are numerous brass "strainer heads" through which the filtered water passes into a system of wrought-iron collecting pipes. These pipes are connected to a large, central, cast-iron collector which passes through the tank and joins the effluent pipe outside. When the sand is to be washed, water is forced backward through the strainers, and at the same time the sand is stirred up to its full depth by means of long iron fingers reaching into the sand and which are attached to a transverse arm mounted on a vertical shaft, the whole being rotated by means of suitable gearing. The agitation and upward flow of water thoroughly cleans the sand in a few minutes. The waste water escapes into a circular trough supported around the inner edge of the tank, and thence passes to a waste pipe. In this particular form, a lower waste is also provided to assist in washing the surface of the filter by surface agitation and drainage from the top, but without reverse flow. Suitable regulating valves are provided to maintain a constant level of water on the filter and a uniform rate of filtration. Each strainer consists of a perforated bronze plate attached to a cylindrically shaped strainer head. These strainer heads are screwed into the branch pipes which form the manifold system. Further details of strainers are illustrated in section 140.

Instead of mechanical agitators, compressed air may be used for agitating the sand, the air being forced through the strainers alternately with the wash water. Mechanical agitators of the type illustrated require the use of circular tanks, while compressed air is readily adapted to any form. In another form of commercial filter, called the "pressure" filter, the entire bed is enclosed in a cylindrical steel tank and the sand is agitated by compressed air.

In many of the modern plants, especially those of large size, the tanks are made of concrete, usually rectangular in form, mechanical agitation not generally being employed. In this case the special devices usually include only the strainer system and the controllers; and several plants have been built where these parts have been furnished by special manufacturing companies, other portions of the plant being designed and constructed independently. Compressed air is used for agitating the sand. During the washing process the dirty water is carried off by means of troughs leading to a gutter and thence to a waste pipe. Convenient piping arrangements are an important feature of such plants. (See section 140.)

137. Operating Principles and Results. The action of rapid sand filters is somewhat unlike that of slow sand filters, although the results are not greatly different. The effect of a coagulant in gathering the sediment into relatively large masses has been explained in section 106. It aids filtration in this way, and also forms a substitute for the organic coating on the sand grains and on the surface of the ordinary sand filter. It is the use of a coagulant which enables such high velocities to be employed. To avoid too frequent washing, it is common to employ heads as high as 10 or 12 feet, but with such high heads and velocities the sand becomes clogged to a considerable depth. The methods of washing, however, readily remove this sediment. The washing process takes 10 to 15 minutes and the interval between washings, i. e., the "run", is 24 hours or less.

In the design of a rapid-filter plant, the preliminary treatment of the water is often a question of much importance if the most efficient methods are to be used. Very turbid waters can often best be handled by permitting a considerable period of subsidence in large basins, followed by coagulation with a further short period of settling; others may require the use of a coagulant at both periods. Many waters not too turbid can be handled by a single brief period of sedimentation accompanied by coagulation. For effective filtration complete clarification is not desirable as the flocculent precipitate is necessary to secure good results in the filter. In many of the early plants the regulation of the rate of filtration and the quantity of chemical applied was very poorly done, but in the later designs very efficient devices have been introduced to accomplish these objects. In the removal of color the rapid filter is advan-

tageous because of the accompanying use of a coagulant. Brown and peaty waters are quite markedly improved in the process.

Extensive experiments conducted at Pittsburgh, Cincinnati, New Orleans, and elsewhere, and results of regular operation indicate that, when rapid filters are properly operated, turbidity can be practically all removed, including a large percentage of color and a considerable portion of dissolved organic matter. Bacterial results are also, in general, as good as those obtained by slow sand filters.

To obtain uniformly good results with economy requires very careful operation. The coagulant must be closely regulated to correspond with the quality of the water; in the case of waters low in alkalinity this is particularly necessary. The efficiency depends so entirely upon the control of these matters that the operation of a rapid filter involves greater care on the part of the attendants than that of a slow filter. It is fully as important in this case also that the whole plant should be under control of bacteriological tests, regularly and frequently made. Many operating points, such as period between washings, wasting of water after washing, exact amount of coagulant needed, can be learned only from experience.

Considering the economic advantages of rapid filters, it may be said that they are especially adapted to those cases where the cost of land is high, where the water is so turbid as to require large settling reservoirs or the use of a coagulant, and in small plants where the unit for slow filters would be very small. They are also well adapted for the rapid removal of iron from ground waters, or of the precipitate in softening plants. (See section 151.)

DETAILS OF CONSTRUCTION AND OPERATION OF RAPID FILTERS

138. Layout of Plant. In a complete rapid-filter plant the essential elements are: the coagulating and settling basin and appliances; the filters; and the clear-water reservoir. In addition to these there may be preliminary settling reservoirs in the case of a water carrying large quantities of sediment. Such reservoirs would usually be constructed quite separate from the filter plant, and, as regards details, need not be considered here. The coagulating basin and the clear-water reservoir may likewise be arranged independently of the filters, but usually one or both are built, with the

filters, into a single structure forming the purification plant. Inasmuch as the coagulating basin constitutes a necessary and vital part of a rapid filter plant, and requires as close attention as the filters themselves, it is especially important that the appliances for operating the basins and the filters be under the same roof and conveniently arranged for operating purposes. The clear-water reservoir, requiring little attention, may be located at any convenient point near at hand.

The best arrangement of parts will depend much upon local conditions. A convenient arrangement of filter units, especially if rectangular in form, is similar to that for large, slow filters; that is,

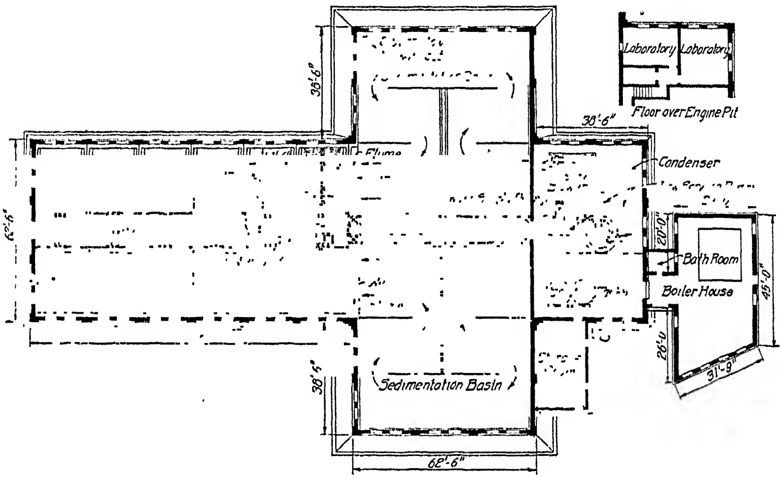


Fig. 46. Plan View of Filters and Coagulating Basin at Youngstown, Ohio
Courtesy of Engineering Record

to place them in two or more rows side by side, with the necessary piping, valves, etc., in a gallery between the rows. The coagulating basin may be conveniently located adjoining the filters, and the clear-water reservoir separately, or, as is quite common, immediately underneath the filters. The introduction of reinforced concrete makes this arrangement economical and satisfactory.

Fig. 46 illustrates the arrangement of filters and coagulating basin at Youngstown, Ohio. The whole is under roof and represents a convenient and compact plan. The clear-water reservoir is located at some distance from the filter plant. The filter units are

14 feet 6 inches by 21 feet in size at the sand level, and between the rows are the pipe gallery and operating platforms. Details of the filter unit and pipe system are shown in Fig. 47.

The most important units of construction include: the sand bed; the strainer system and collecting pipes; the agitating system;

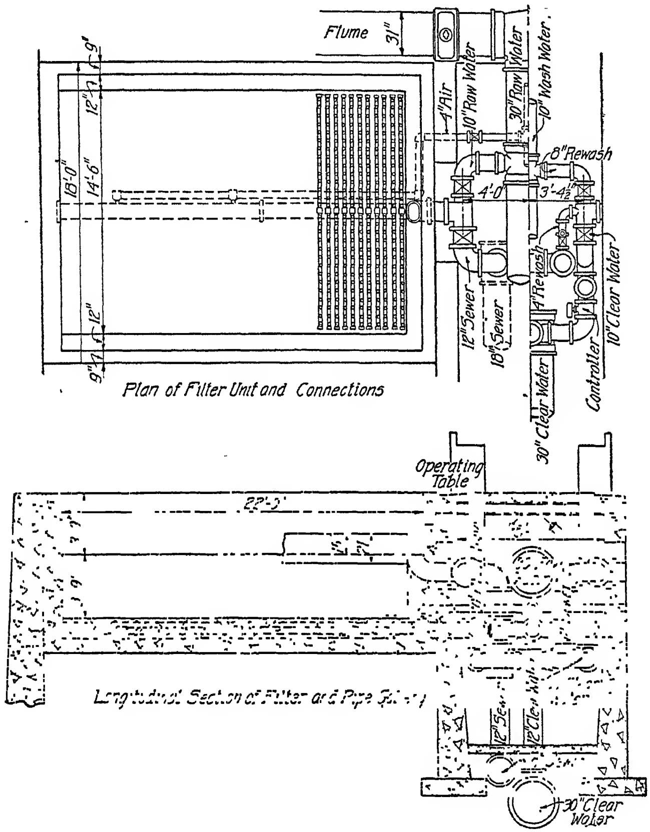


Fig. 47. Plan and Longitudinal Section of a Filter Unit of the Plant at Youngstown, Ohio
Courtesy of Engineering Record

the wash-water appliances; means for controlling the rate of filtration; the coagulating system, including means for preparing solutions and regulating their application, and the arrangement of coagulating basins; arrangement of piping and valves; and various other devices for operating the plant.

139. Sand Bed. Experience has shown that the most satisfactory sand is one of quite uniform grain and of an effective size of from .3 to .4 mm., the best size depending somewhat upon the character of the water to be treated. It should be quite uniform in size, as a sand consisting of large and small particles will tend to stratify in the washing process. A depth of 30 to 36 inches is usually employed. The sand bed is supported on a layer of fine gravel, 6 to 8 inches or more in thickness, which permits the perforations in the strainers to be of fairly large size. This gravel should be carefully screened and of as large a size as practicable without allowing the superimposed sand to penetrate into the pore spaces. Usually two or three grades are employed, the upper one being about .05 to .10 inch in size, and the lower one about $\frac{1}{4}$ to $\frac{1}{2}$ inch. The gravel should be free from fine materials, and as uniform as possible in order to avoid being disturbed in the washing process. Crushed and screened quartz is often used for the sand and gravel, but natural materials well screened are equally satisfactory.

140. Strainer System and Collecting Pipes. The design of the strainer and collecting system is a matter of greater difficulty than in the case of the slow sand filter. As in that type, the collecting system must, first of all, be sufficiently extensive to cause the total loss of head to be nearly uniform over the entire area.

The strainer system must serve also to distribute the wash water uniformly into the sand bed. To accomplish this there must be a considerable resistance to flow through the strainer, as compared to that through the pipe system, so that, as the water forces its way through the sand, a considerable reduction of resistance in the sand at one point will not materially change the pressure at other points. This requires the strainer openings to be small, numerous, and well distributed, and the pipe system to be relatively large and arranged so as to give practically equal pressures at all points. The collecting pipes must be designed with reference especially to the amount of wash water required and must be arranged in units of not too large size. The unit of area served by one collecting main is commonly made from 10 to 15 feet wide by 15 to 20 feet long. In large plants it is convenient to group together from 2 to 4 such units to serve a single tank, the size of tank depending much upon the total size of plant.

In Fig. 48 is illustrated a common arrangement of collecting pipes and strainers. In this case the effluent pipe connects with a large central cast-iron collector, or "manifold", lateral collecting pipes, placed about 10 inches apart. Into these are screwed brass strainers which are also spaced about 10 inches apart. These strainers are perforated with numerous small holes. In the earlier practice the holes were very small, but in later plants they are made larger, $\frac{1}{16}$ inch to $\frac{3}{32}$ inch being common. Large holes are less apt to clog up, and, with the use of gravel beneath the sand, are much more satisfactory.

Fig. 48 shows the arrangement used in the Cincinnati plant. The laterals consist of concrete channels 12 inches apart, with

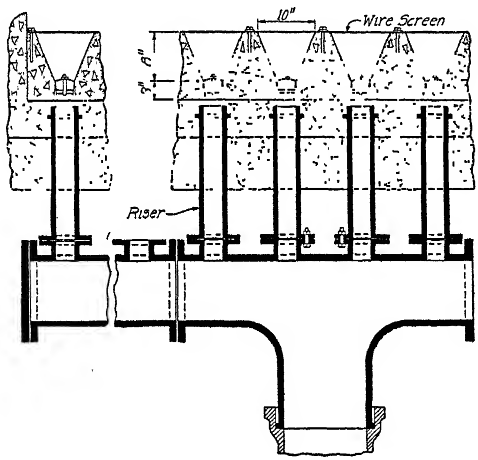


Fig. 48. Section Showing Details of Strainer for the Filtering Plant at Cincinnati, Ohio
Courtesy of Engineering Record

strainers of long, brass plates, perforated with sixty-four $\frac{3}{32}$-inch holes per lineal foot. From each of these lateral channels, connections are made by means of $3\frac{1}{2}$-inch cast-iron risers to the main collector located beneath the floor. The furrows in the concrete floor are 8 inches deep and contain all the gravel. Above this gravel is placed a wire screen of No. 20 brass wire, having 10 meshes per inch. The purpose of this screen is to hold the gravel in place during washing, the water pressure employed here being especially high, as no other method of agitation is used. The units of the

collecting system are $12\frac{1}{3}$ feet by 14 feet, and four such units make up one tank unit. The filter unit comprises two such tanks.

141. Agitating System. Two general methods of agitating the sand are in use: the mechanical agitator; and agitation by compressed air. The usual form of the mechanical agitator is illustrated in Fig. 45. It consists of deep rakes which are moved through the sand during the washing process. After the operation is complete the rakes are lifted out of the sand. While the circular form of tanks is better adapted to the use of this form of agitator it has been used to some extent with rectangular tanks, but compressed air is much more convenient in that case.

In the use of compressed air, the air, under a pressure of 3 to 5 pounds, is forced through the sand bed, either through the strainer system itself or through a separate pipe system located in the gravel just above the strainers. In the air-distributing system, as in the water system, it is necessary to use such an area of openings as to admit the desired amount of air and at the same time to offer a considerable resistance to exit as compared to the resistance in the pipe system. These requirements make it necessary to use a much smaller total area of openings than in the water system, an area of .02 to .03 of a square inch per square foot of filter being common. Where the air is forced through the strainer system an ingenious arrangement of the strainer traps off the main opening so that the opening for air is reduced to the desired amount. In this case the air pipes are connected at intervals with the collecting main, through which the air passes to the trapped strainers. In the operation of washing, air and water are used alternately.

In some plants water alone is successfully used without other means of agitation, notable examples being the large plants at Cincinnati and New Orleans. In these plants no provision is made for special agitation, but the wash water is used under relatively high pressure. Experience in some plants indicates that the results gradually deteriorate if water alone is used, although at first there appears to be no difference.

142. Wash-Water Appliances. The wash-water appliances consist of the pipe connections, arranged so that the flow may be reversed in direction, the agitating system, and the means for taking off the dirty water from above the filter. The wash water is supplied

under a pressure of about 10 to 15 pounds either by means of suitable pumps, or from the high-pressure system through reducing valves, the former being generally the more economical method. It is admitted to the effluent pipe of the filter through suitable pipe connections and controlling valves. The dirty water from the filter passes through troughs or gutters built across the bed or at the margin, and thence is conducted through pipes to the drain. In order readily to carry off the wash water, the gutters need to be spaced so that the lateral movement of the water is relatively small, not more than 3 to 4 feet. These gutters are conveniently built of reinforced concrete. They are placed about 1 foot above the surface of the sand bed. Generally the unfiltered water is admitted through the same pipe connection that serves as the outlet for the wash water, the gutters thus serving as weirs to distribute the raw water until the filter is partially filled. After washing a filter it is sometimes desirable to waste the effluent for a time. Provision should always be made for this by constructing suitable connections from the effluent pipe to the drain. The time required for washing is only 10 to 12 minutes, and the amount of wash water required is usually from 4 to 5 per cent of the total amount filtered.

143. Head Employed and Manner of Controlling Filtration Rate. At the ordinary rate of 100 to 125 million gallons per acre per day, the minimum frictional resistance is usually from 2 to 3 feet. As in the slow sand filter, arrangements must be made whereby, as the filters become clogged, this loss of head may be increased up to the maximum desirable amount, and so varied as to maintain a uniform rate of filtration. The maximum loss of head is generally made about 10 or 12 feet. A greater maximum tends to increase the cost of plant and the penetration of suspended matter into the filter, while a lesser maximum tends to increase the frequency and cost of cleaning and the proportion of area out of service. With this maximum available head the period of service will usually range from 6 to 12 hours.

The head is controlled in a manner similar to that used with slow filters, that is, by maintaining a constant level of water on the filters and varying the pressure head in the effluent pipe by some means more or less automatic. Generally automatic controllers are used, a controller being inserted in the effluent pipe of each filter unit.

The means employed to maintain a constant level of water on the filter is the usual balanced valve placed on the entrance pipe and regulated by a float. A butterfly valve is also often employed for this purpose. Generally the level of water in the entire system of filters is maintained at a uniform elevation, and equal to that in the coagulating basin, so that the regulating valves are placed in the basin only. The most suitable arrangement depends upon local conditions.

144. Coagulating System. The various parts of the coagulating system are the same as described in section 114. The period of coagulation is important, as the success of the plant depends much upon this feature. In the early plants this was very inadequate, being but a few minutes. Generally a period of 3 to 6 hours is now provided, the most advantageous period depending upon the character of the water. Too perfect sedimentation is undesirable as it removes too fully the coagulum upon which the efficiency of the filter depends. The coagulating basin of the Youngstown plant is well shown in Fig. 46. Here the effluent passes over a broad weir into the pipes leading to the filters.

145. Arrangement of Piping System. The pipe system includes the following: inlet pipes for the raw water; outlet pipes for effluent; pipes for wash water; pipes for wasting dirty water to the drain; pipes for wasting effluent; and, generally, air pipes. The large mains of these various systems are generally placed in a pipe gallery between rows of filters and branches taken off at each filter as shown in several of the illustrations. In the large units at Columbus and Cincinnati the filter unit is divided in the center by a horizontal gutter into which the raw water is discharged, and which also receives the dirty water in washing. The air-pipe connections are also made by means of branches taken off in the central channel. The branch pipes from the raw-water main and from the waste-water main usually connect to the same point in the filter; likewise the branches from the effluent and from the wash-water mains. An additional cross connection is placed between the effluent pipe and the waste pipe to permit of wasting the effluent.

146. Other Devices Used in Plant Operation. Besides the features already described, other details which require careful attention are the various devices used in the operation of the plant. For

the operation of the valves hydraulic pressure is generally employed, the pressure pipes being all operated from tables on the operating platform. Loss-of-head gages and water-level gages should be provided, as in a slow-filter plant, also convenient means for sampling. Proper laboratory facilities for the study and control of operation are important.

COST OF RAPID FILTERING PLANTS

147. First Cost. The cost of a rapid filter plant under ordinary conditions will range from $8,000 to $12,000 per million-gallon capacity, for filters, coagulating basin, clear-water well, and auxiliary pumping apparatus.

148. Operating Cost. The cost of operation is largely dependent upon the amount of coagulant used. Compared to the cost of sand filters the first cost will usually be less, but if much coagulant is used the cost of operation will be more. Which system is the more economical thus depends upon the character of the water treated and other local conditions. Under ordinary conditions the cost of operation per million gallons will range from $4 to $6; and, including capital charges, interest, and depreciation, the total cost will range from $10 to $12 per million gallons.

MISCELLANEOUS METHODS OF PURIFICATION

149. Special Forms of Filters. Besides the two principal types of sand filters discussed in the preceding chapters, various special forms of sand filters and filters composed of other materials have been employed to a limited extent.

A filter made of porous, artificial stone, known as the "Fischer system", is used to some extent in Germany. The stone slabs are set up edgewise and fastened together in pairs forming hollow cells into which the water is forced by pressure.

Another form of rapid filter known as the "Maignen scrubber" is used to a considerable extent as a preliminary filter. It is composed of layers of coarse gravel and slag covered with a layer of compressed sponge. The water enters at the bottom and flows upward, the rate being ordinarily about 60,000,000 gallons per acre per day. Generally about 60 per cent of the turbidity and 75 to 80 per cent of the bacteria are removed. The action is partly

sedimentation and partly filtration. Scrubbers of this type are used at Philadelphia and also at South Bethlehem. At the latter place the coarse material consists partly of gravel and partly of coke. Inclined layers of slabs are placed in several of the upper rows to act as deflectors to aid sedimentation. The scrubber is cleaned by reversing the flow of water. The sponge is also removed and washed occasionally and, if necessary, the coke may be treated in the same way. The rate of filtration through the scrubbers is 28,000,000 gallons per acre per day and 7,000,000 through the sand filters.

150. Aeration. Attempts often have been made to purify water of organic matter by aeration. The presence of oxygen is certainly necessary for the action of the nitrifying organism, but to add large quantities of oxygen to water that already contains oxygen appears, from analyses of aerated water in various places, to have little or no effect on the organic matter.

Though aeration may effect little or no change in the organic matter present in a water, it does have a very important action in the case of waters from ponds and reservoirs which possess offensive odors or tastes because of certain dissolved gases present. These gases may arise either through the putrefaction of dead organic matter, such as the vegetation left in a reservoir when constructed, or the dead algae and other organisms which may periodically grow in the water, or they may be formed during the growth of certain microscopical organisms. In any case aeration is very effective as it causes the displacement of the objectionable gases by the gases of the atmosphere.

In the removal of iron from ground waters, aeration also plays an important part, as more fully described in section 152.

Aeration is accomplished in various ways. It may be effected by causing the water to flow over cascades or weirs, or to fall freely from broad areas of perforated plates, or by still other means. The more extensive the aeration required, the more thorough must be the exposure to the air.

The benefit of aeration explains why a well water raised by buckets is more commonly free from bad tastes and odors than where a pump is used, although such odors and tastes are not in themselves dangerous to the health.

151. Softening Water. Water is rendered hard by the presence of lime and magnesia, chiefly in the form of carbonates and sulphates, but occasionally as chlorides and nitrates. The carbonates cause so-called temporary hardness (removable by boiling), while the sulphates and other compounds cause permanent hardness. In using a hard water for washing purposes approximately 2 ounces of soap are neutralized or wasted for each 100 gallons of water for each grain per gallon of calcium carbonate or its equivalent. In boiler use the carbonates of lime and magnesia are precipitated, forming a deposit which can usually be removed by blowing out, unless accompanied by scale-forming substances. Sulphate of lime precipitates at high temperatures and forms a very hard, objectionable scale, particularly if the water contains other suspended matter. The softening of water is, therefore, of great economic importance.

The softening of water is accomplished by simple processes of chemical precipitation. To remove the carbonates, lime is used as the precipitant. The carbonates are held in solution chiefly by virtue of the carbonic acid dissolved in the water, and on adding lime the acid unites with it, forming carbonate of lime. In the case of hardness due to the carbonate of lime the reaction is

$$CaCO_3 + CO_2 + Ca(OH)_2 = 2CaCO_3 + H_2O$$

The resulting carbonate is now but slightly soluble and so precipitates out. The $CaCO_3$ (lime carbonate) and the CO_2 (carbonic acid gas) are present in the water; the $Ca(OH)_2$, ordinary lime, is the chemical added.

To remove the sulphate, sodium carbonate (Na_2CO_3) is used. The reaction is

$$CaSO_4 + Na_2CO_3 = CaCO_3 + Na_2SO_4$$

The carbonate of lime precipitates out as before while the sodium sulphate (Na_2SO_4) is not especially objectionable. Various methods of carrying out the details of the process, relating principally to the application of the chemical and the removal of the precipitate, have been devised. These are known under various names, but the general principle is the same in all. The lime is usually added in the form of lime water, a solution of slaked lime in water.

In general the water to be treated is run into large tanks, the chemical added and then the precipitate allowed to settle as far as

practicable. The water is then drawn off and the remaining precipitate removed by rapid filtration. In purifying water for boiler use the precipitate can be removed to a sufficient extent by the use of settling tanks alone. The chemicals used are lime and usually soda ash, or crude sodium carbonate. One of the largest softening plants is that at Columbus, Ohio, with a daily capacity of 30,000,000 gallons. Both the carbonates and the sulphates are removed.

Many scale preventives have been proposed for use in boilers, but probably the best in general use is sodium carbonate. This breaks up the sulphates as previously shown, and thus prevents the formation of a hard deposit; but the precipitation of the carbonates is increased by the process. The sodium sulphate remains in solution, but should not be allowed to concentrate too greatly or it will cause foaming.

152. Removal of Iron from Waters. Ground waters may not infrequently contain iron in solution and so have their taste and appearance impaired. Such waters are likely to be not only disagreeable to the taste, but objectionable for domestic use, especially in the laundry. Waters containing iron are clear when first drawn, but soon become cloudy on standing, due to the absorption of oxygen from air and the consequent conversion of the soluble ferrous salt into ferric hydroxide. This material in time settles out as a rusty precipitate. Sometimes, where there is an abundance of organic matter in solution, as in waters from peaty sources, soluble compounds are formed with the organic matter that are not readily oxidized upon exposure to air.

In many cases where the iron is present as ferrous carbonate, it can be removed by oxidation if exposed to the air. This reaction is utilized in the practical treatment of such waters, and in most of the plants installed for the removal of iron from ground waters aeration is employed to facilitate this oxidation. The precipitated iron (Fe_2O_3) is generally removed from the water by rapid filtration through sand.

The extent of aeration required varies considerably, according to the character of the water, and the conditions necessary for successful treatment cannot in all cases be determined without experiment. In some cases simple exposure in open canals gives sufficient aeration, or the mere spraying in small jets, or other simple means

may be successful. In some cases the difficulty of aeration is probably due to excess of organic and of free carbonic acid. At Reading, Massachusetts, lime and sulphate of aluminum have been successfully used in connection with aeration and filtration. This process, however, increases the hardness very considerably.

STERILIZATION AND DISTILLATION

153. Ozone. Various methods of sterilizing a water by means of minute quantities of chemicals have been developed in recent years and are of much importance, especially in case of sudden outbreaks of disease or temporary disturbance of a water-purification plant. Some of these methods have come into common use as a process applicable to a clear water with a small amount of dangerous pollution, or as a supplement to filter plants.

Of these, the ozone method, used already in certain plants, is very promising. Ozone gas is a powerful disinfectant and is readily absorbed by the water when sprayed into the air. It is a relatively expensive process and of less general merit than the process next mentioned.

154. Hypochlorite of Lime. The use of this disinfectant for destroying the bacteria in a water supply has become very common. This chemical is an exceedingly strong germicide by reason of the active chlorine it contains. Water so treated is perfectly harmless, in fact remains unchanged except for a slight increase in hardness. The cost of chemicals will range from 10 cents to about 40 cents per million gallons treated.

The hypochlorite treatment is used in several large cities as the sole treatment, the water being naturally free from sediment. It is used in connection with filter plants in many cities, among them being Cincinnati, Ohio; Davenport, Iowa; and Terre Haute, Indiana. So effective is the process that portable plants have been constructed by several state boards of health to use in case of outbreaks of water-borne diseases. From 5 pounds to 20 pounds of chemical are used, giving from .1 to .5 part of available chlorine per million gallons of water. Excessive amounts are likely to give a slight taste to the water and should be avoided.

The efficiency of the treatment is very high. Combined with filtration it will remove nearly all the bacteria, the percentage

removal commonly reaching 99.9. The average results at Cincinnati for the year 1914 were as follows:

	Bacteria Present (per c.c.)
River water	16,500
Settled water	3,570
Filtered and treated	36
Total efficiency (per cent)	99.78

During the months of January to May, hypochlorite was added to the filtered water. The average number of bacteria in the filtered water during this time was 226 per c.c., and in the water after hypochlorite treatment only 37 per c.c.

155. Copper Sulphate. The action of copper sulphate as a germicide is well known, and its use for this purpose has been more or less studied, but it has been generally objected to on account of its possible deleterious effect on the human system. Its use to destroy and prevent the growth of objectionable algae and other microscopical organisms in reservoirs is of much more importance and has been successfully applied in many cases. At Hanover, New Hampshire, a reservoir of 100,000,000 gallons received a single treatment, using a proportion of 1 part in 4,000,000. The number of micro-organisms was decreased in 24 hours from 600 per c.c. to 60 and wholly eliminated in 60 hours.

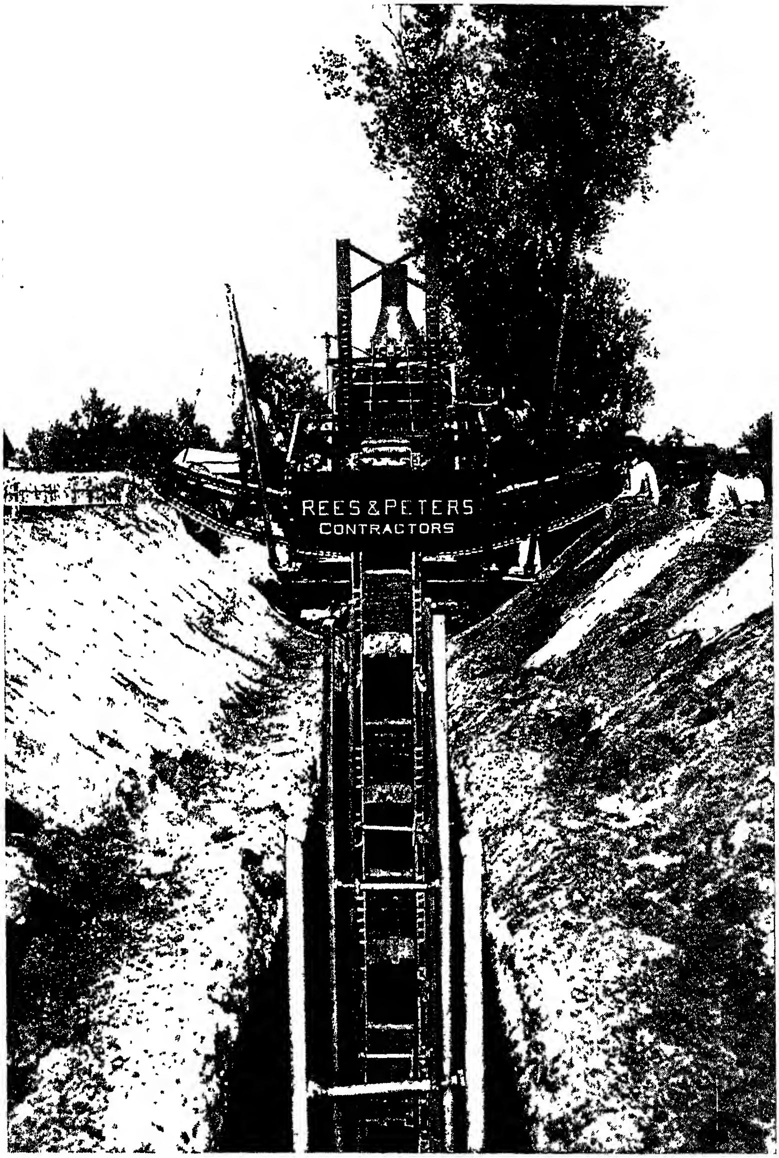
It is found in practice that an amount of copper sulphate of 1 part in 2,000,000 is sufficient to destroy most of the objectionable forms of organisms, some being rapidly destroyed with an application of only 1 part in 20,000,000. In these minute quantities no harmful effect can arise from its use in a drinking water, and considering that very few applications are needed during the season and that a large portion of the copper is precipitated with the organisms, there would seem to be no objection to its use under proper supervision. The method of application which has been frequently employed is to drag sacks containing the copper sulphate back and forth through the reservoir or pond in a more or less systematic manner. With careful manipulation this method will serve to distribute satisfactorily the desired amount of material, but at the best it would appear that some form of spray apparatus using a definite solution would be more satisfactory.

156. Boiling. Another method of purification on which even greater reliance can be placed than on the use of a chemical is the use of heat. There are no pathogenic bacteria that are liable to be distributed by the way of the water supply that are able to withstand the influence of boiling water for a period exceeding 10 to 15 minutes. Cholera and typhoid succumb in 5 minutes or less. In case of sudden outbreaks of disease or temporary disturbance of installed water supplies, this method can always be relied on with perfect safety. Boiling does not, however, render potable a water containing large amounts of organic matter, although it may destroy the disease germs that may be therein. By distillation a water can be obtained free from dissolved matter as well as bacteria. This process is extensively used on shipboard to obtain potable water from sea water, and in a few places on the seacoast for similar purposes.

157. Domestic Filters. Frequently it is advisable to purify water supplies for household use. For this purpose a large number of different filters have been devised, but many of these are so inefficient as to be worse than useless; for it not infrequently happens that the possession of a filter lulls the consumer into a state of false security. The best of these filters suitable for household use are those that are made of unglazed porcelain (Pasteur filter) or fine infusorial earth (Berkefeld filter). Both of these filters deliver a wholly germ-free filtrate when they are first put in service, but unless close attention is given them they sooner or later lose this property. Generally speaking, these filters should be cleaned and sterilized in boiling water or in steam under pressure once a week in order to kill out the germ life that has found lodgment in the pores. In this way not only is the sterility of the filtrate maintained, but the yield of filtered water is increased.

Filters of this class are not often used for municipal purification, but are admirably adapted for schools, garrisons, prisons, or hotels, as well as for private use.

Other types of household filters, such as those constructed of porous stone, charcoal, or asbestos, have been on the market for many years. Judged from the popular standpoint of purity, which is generally the production of a clear water, many of the filters would be regarded as quite satisfactory; but as a means of removing germ life they possess for the most part but little merit.



PARSONS TRENCH EXCAVATOR CUTTING A 15-FOOT DITCH IN A 14½-FOOT ALLEY

Courtesy of G. N. Parsons Company, Newton, Iowa

SEWERS AND DRAINS

PART I

1. Introductory Definitions and Discussions. *Sanitary Engineering* is that branch of engineering which has to do with constructions affecting health. It thus might be claimed to include the manufacture and transportation of foods, the architecture of buildings, and many other things which affect the health of communities; but in ordinary use, a more restricted definition of the term is adopted.

In common practice, the term *Sanitary Engineering* is taken to include only *water supply engineering* and *sewerage engineering,* the former branch dealing with securing a satisfactory supply of water, and the latter with the satisfactory removal of surplus and waste liquids. Sewerage is the subject of this instruction paper, water supply being treated by itself.

Sometimes *sanitary engineering* is given a still more restricted meaning, and is taken to include sewerage only.

A *drain* is a canal, pipe, or other channel for the gradual removal of liquids. In sanitary engineering, the two principal kinds of drains are, first, those for the removal of comparatively pure ground waters and surface waters, as in land drainage; and, second, those for the removal of polluted liquids, as in sewerage systems.

A *sewer* is a drain for the removal of foul, waste liquids. Usually sewers are closed, underground conduits. An *open sewer* is an open channel which conveys foul, waste liquids.

Sewerage is a general term referring to the entire system of sewers, together with any accessories, such as pumping plants, purification works, etc. Thus we may speak of the "sewerage" of a city, or of the "system of sewerage," or of the "sewerage system."

Sewage is any foul, waste liquid.

Sanitary sewage is the foul wastes of human or animal origin from residences, stables, stores, public buildings, and other places of human or animal abode. By far the greater part (usually 99.8 per cent or more) of sanitary sewage, commonly, is ordinary water, which

is added to the wastes themselves in this large volume simply to facilitate removal.

Manufacturing sewage is the foul wastes from factories. In different factories, it is of extremely different nature. It is often exceedingly strong, and very offensive and difficult to dispose of, as compared with sanitary sewage.

Storm sewage is the storm water flowing from city surfaces during and after rainstorms. Though polluted, especially at the beginning of a storm, from the droppings of animals and the other surface filth of cities, it is not so foul, nor so liable to swarm with disease germs, as is sanitary sewage.

The terms *sewage* and *sewerage* are often misused by persons not engineers, to mean the same thing. Thus such persons often speak of the "sewage system" instead of the "sewerage system;" of the "disposal of the sewerage" instead of the "disposal of the sewage," of a city. So common is the misuse that some sanction can be found in the dictionaries; but engineers should be careful to restrict the meaning of the word "sewage" to the liquid which flows in the sewers, while the word "sewerage" should never be so applied.

Sewer air, often miscalled *sewer gas*, is the air in the sewers above the liquid contents. It has no definite chemical composition, but contains varying proportions of pure air and of carbonic acid gas, marsh gas, sulphuretted hydrogen, and the various products of decaying organic matter. Sewer air is constantly changing in composition even in the same sewer. While considered injurious to health when breathed, it has not been proved to be in itself the direct means of communicating infectious diseases.

2. Historical Review. Sewers and drains are of very early origin. Among the ruins of all ancient civilizations, are found the remains of masonry and tile conduits constructed for drainage purposes.

In Fig. 1, for example, (from Fergusson's *History of Architecture*), are shown the remains of a large masonry sewer or drain built by the ancient Assyrians in the eighth or ninth century B. C., for one of their palaces at Nimrud. This is one of the earliest examples found of the use of the arch in masonry.

In Fig. 2 is shown the mouth of the *Cloaca Maxima*, or great sewer, of ancient Rome, built in the seventh century B. C., and still

in use after the lapse of 2,500 years. Without this sewer, a large tract of ancient Rome could not have been inhabited; and in speaking

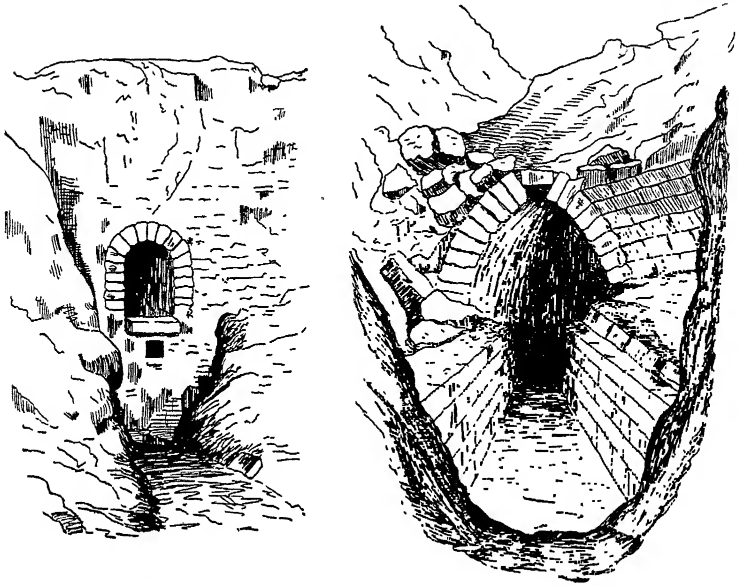


Fig. 1. Ancient Assyrian Sewers at Nimrud.

of it, one authority says: "To this gigantic work, admired even in the time of the magnificent Roman Empire, is undoubtedly owing the

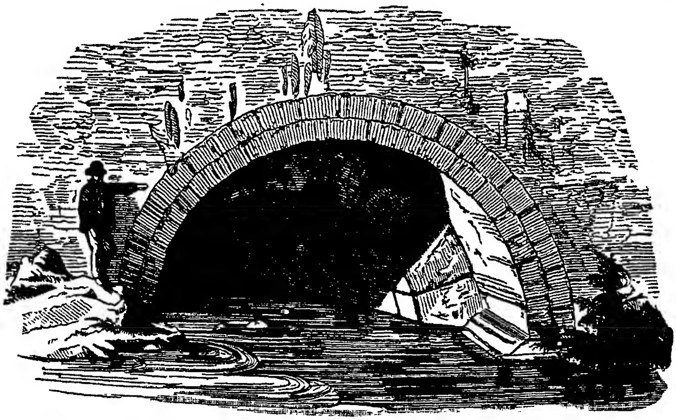


Fig. 2. Mouth of the *Cloaca Maxima*, or Great Sewer, of Ancient Rome.

preservation of the Eternal City, which it has secured from the swamping that has befallen its neighboring plains."

In many other ancient cities and structures, the remains of intelligently planned drainage systems have been discovered; and it is evident that the ancients paid great attention to this matter so vitally affecting health. The art reached its highest ancient development in the time of the Roman Empire. The Romans, in fact, were the greatest engineers of antiquity, and especially excelled in sanitary engineering (both water supply and drainage). They were proficient in land drainage, as well as in sewerage.

With the fall of the Roman Empire, sanitary engineering suffered the same retrogression which befell learning and science; and for a thousand years—throughout the Middle or Dark Ages—it was almost entirely neglected. The impure water supplies and the accumulated filth of mediæval cities produced fearful consequences in the terrible pestilences which desolated Europe.

With the revival of learning and science in the 14th and 15th centuries, attention again came to be paid to sanitary engineering; but for three or four hundred years more, little was done toward putting drainage and water supply on a scientific basis. Drains, rather than sewers, were built in the various towns as absolute necessity made imperative; but they were constructed piecemeal, and not so as to form comprehensive systems. They were not made watertight or self-cleaning; but it was usually considered necessary to make them large enough for men to enter to remove the filth, whose accumulation and festering in them were believed unavoidable.

In England, modern sanitary engineering may almost be said to have had its origin; yet so late as 1815, laws were enforced forbidding the emptying of fæcal matter into the sewers. "Such matter was generally allowed to accumulate in cesspools, either under the habitations of the people or in close proximity thereto." * In fact, though no longer enforced, these laws were not repealed until 1847, when Parliament passed an exactly contrary act, making it compulsory to pass fæcal and other similar foul matter into the sewers.

Modern sanitary engineering, especially as regards sewerage and drainage, has had almost its entire development since 1850. It was not until 1873 that there was published a comprehensive treatise on sewerage, that of Baldwin Latham, already quoted. At about this time, also, much attention began to be paid in England to sewage

*Baldwin Latham.

purification. It was reserved, however, for America to put sewage purification on the road to a satisfactory scientific solution, by the thorough investigations of the Massachusetts State Board of Health, begun in 1887 and still under way.

In America, much was done in the third quarter of the 19th century to advance sewerage engineering, through the studies of able engineers in connection with the design of systems for Chicago, Brooklyn, and other large American cities, the results being published in papers and reports, or in book form.

About 1880 the *separate* system of sewerage came strongly into prominence in America, as advocated by the late Col. Geo. E. Waring; and the construction of the Memphis (Tenn.) sewers on this system at that time, together with their great success in putting a stop to the fearful epidemics which had so often desolated that city, did much to make sewerage possible for small cities. At present, sewers have become so common and so necessary in modern life, that villages of 2,000 population, or sometimes of even less, are very generally taking up their construction.

With the present wide adoption of sewers, even by small communities, sewage disposal has come to be of very great importance, and is now undergoing great development. Many discoveries remain to be made in this line, in which the guiding principles have not yet been so thoroughly worked out as in the construction and maintenance of sewers themselves.

3. Importance and Value of Sewerage and Drainage. The importance and value of the constructions of sanitary engineering can hardly be exaggerated. Upon them absolutely depends the health of every city. One needs but to read descriptions of the great modern epidemics of yellow fever at Memphis and New Orleans, or of cholera at Hamburg, or to have been engaged to visit as sanitary engineer an American town during one of the numerous recent outbreaks of typhoid, to understand the truth of the scripture, "All that a man hath will he give for his life." Yet not only could sanitary engineering absolutely prevent every such epidemic; but, in addition, it could annually save thousands upon thousands of other lives which now succumb to bad sanitation.

Already very much has been accomplished in this direction by improved sanitation, though ideal conditions are yet seldom attained.

A prominent sanitary engineer estimated from actual statistics, that as early as 1885 there was a saving from this cause of 100,000 lives and 2,000,000 cases of sickness, annually, in Great Britain, in a total population of only 30,000,000. Figuring on the basis of the money value alone of the lives saved, and of the sickness and loss of time avoided, the money value of the above result would be almost incalculable.

In many individual cities, statistics have shown in death rates an immediate lowering, due to the construction of sanitary improvements, more than sufficient in money value to the community to pay for the entire cost. Funeral and sickness expenses saved, alone, often make enormous sums.

In this connection, it should be said that pure water supply and good sewerage are both essential, and that it is impossible to separate the value of one from that of the other. A polluted water supply may spread disease, no matter how perfect the sewerage, and an abundant water supply is essential to the proper working of sewers. On the other hand, without sewers and drains, an abundant water supply serves as a vehicle to enable unmentionable filth to saturate more deeply and more completely the soil under a city. Cesspools are even more dangerous than privy vaults.

In addition to direct prevention of communication of disease by unsanitary conditions, modern sewerage facilities are so great a *convenience* that this advantage alone is usually more than worth the cost. This is shown by the increased selling and rental value of premises supplied with sewerage facilities. No sooner is a partial or complete sewer system constructed in a town, than prospective buyers or renters begin to discriminate severely against property not supplied with modern sanitary conveniences; and persons looking for new locations for business ventures or residence purposes, discriminate in like manner in favor of towns having good sewerage.

So great has become the demand for sanitary conveniences, that they are now being installed in farmhouses as well as in the city. It is now possible for any farmer, at an expense of only a few hundred dollars, to have hot and cold water piped under pressure in his house, a bathroom and other plumbing fixtures, and his own sewage-disposal plant. This has already been accomplished in many cases. Such improvements, if made in accordance with correct principles, greatly

better the sanitary conditions of the home; and they also prevent much disease by doing away with the exposure to inclement weather, which is so dangerous an accompaniment of the old-fashioned, barbarous, outdoor privy.

The great importance of sewerage may be realized by giving some consideration to the enormous sums of money which have already been spent for sewer systems in this country alone. Villages of 3,000 population in rural communities, often spend $50,000 or more upon a system. The city of Chicago has in recent years spent $50,000,000 in securing merely a satisfactory outlet for its sewers, without counting a dollar of the vast sums expended on the sewers themselves. In the United States, hundreds upon hundreds of millions of dollars have been invested in sewers.

SYSTEMS OF SEWERAGE

4. A *privy vault* is a receptacle, usually a mere excavation in the ground, for the reception of fæcal matter and urine. To prevent dangerous pollution of the surrounding soil and ground water, privy vaults should be lined with water-tight masonry; but this is seldom attempted, and even if attempted, is still more seldom accomplished, for it is difficult in such work to secure absolute freedom from leakage. The privy vault, frequently, is simply abandoned and covered over with earth when full, it being cheaper to change the location than to clean out the old pit.

The privy vault, with its inevitable befouling, in the immediate vicinity of the home, of earth, air, and water, the three great requisites of health, and with its danger from pneumonia and other diseases which may be contracted from exposure, should be adopted only in case of absolute impossibility to secure something better, and even then only as a temporary resort. It is not so objectionable in the country as in the city, if located far away from the well; but here the trouble is that it is usually placed too close to the well which furnishes the drinking water. In the country the leachings from hog pens, cattle yards, and manure piles frequently add to the contamination of the drinking water. It is impossible to set any safe distance at which a well may be placed from a privy, owing to the variable nature of the soil. The contamination may be carried very far in gravel

strata or rock crevices. Impervious clay confines filtration within narrower limits.

5. A *cesspool* is a receptacle for receiving and storing liquid sewage. It consists usually of an excavation dug in the ground, lined with masonry, and covered, into which the sewer from the house discharges. To prevent contamination of the surrounding soil and ground water, the cesspool should be made absolutely water-tight, and its contents should be removed whenever it becomes full.

A *leaching cesspool* is one not made water-tight. The liquid contents partly leach away into the surrounding soil, and often into sand or gravel strata, or crevices in the rock, which may carry the contamination to great distances. Owing to the offensive nature of the work of cleaning out cesspools, and to the expense thereof, cesspools as a usual thing are deliberately made not water-tight. The owner congratulates himself if he strikes a crevice in the rock or a gravel stratum which prevents his cesspool from filling up, though even a little thought will often show that he is thus directly contaminating the water vein which supplies his own or his neighbor's well. Even then he does not usually escape permanently the expense and annoyance of being forced to clean out the cesspool, for in time almost any crevice or porous stratum will clog so as to permit only partial escape of sewage.

Leaching cesspools should be absolutely prohibited by law. They are even more dangerous than the privy, for the liquid sewage in them can penetrate further into the surrounding soil than the fæcal matter of the privy vault.

The frequent effect of cesspools and privies is illustrated in Fig. 3, which does not at all exaggerate conditions very frequently found in cities and villages. Often the tearing down of old buildings, prior to the erection of new, exposes to view the rear of lots, and shows sometimes a half-dozen privies grouped within a few rods of several wells. The nose and the eye give convincing evidence of foulness in such cases; and chemical or bacterial analyses are not necessary to demonstrate the danger in using the wells; but the same dangerous conditions pass unnoticed in many other places in the same city, because not exposed to casual view. In time, the whole ground water under such a village or city becomes contaminated, and poisons wells and damp cellars and the exhalations from the ground.

6. A *dry closet* is a privy having a tight, removable receptacle in place of the vault, and provided with means for covering the contents with dry dust, ashes, or lime each time the closet is used. Usually a small shovel and a box are used to hold the dust or other absorbent material. Enough of the dry material should be used to absorb all liquids. The contents should be removed and hauled away in the tight box when it is full, to be emptied in a safe place or used for fertilizer. The dry earth closet is an improvement over the privy vault, but is not a safe or otherwise satisfactory arrangement.

7. The *pail system* is one in which the fæcal matter and urine are received in tight pails, which are removed daily, or at least every few days, by regular city employees. The pails are carried to some safe place, there emptied, and returned after disinfection. Although the pail system has been tried in America under exceptional conditions, it is entirely unsuited for use here, and is almost never employed, even in Europe, where the people will submit to the police interference necessary for satisfactory operation.

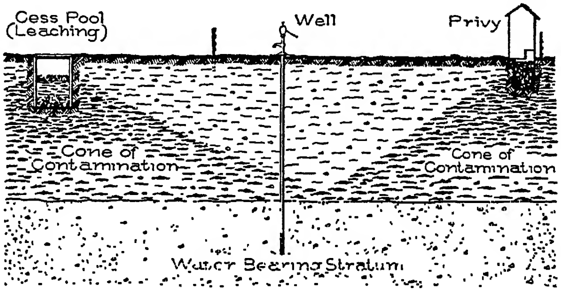


Fig. 3. Showing How Contamination of Well Water may Occur through Proximity of Cesspools and Other Sources of Filth.

8. *Pneumatic systems* of sewerage are those in which the sewage is forced through the street pipes by air, either by a partial vacuum, as in the *Liernur system* (tried in Holland), or by compressed air, as in the *Berlier system* (tried in France). Neither system is used at all in America, or to any important extent in Europe. The expense of construction and operation, and the liability of all such mechanical appliances frequently to get out of order, make them unworthy of consideration.

9. *Crematory systems* are devices for disposing of fæcal matter, urine, and garbage on the premises by drying and then burning. There are several patented methods. The matter to be disposed of is received in a furnace-like structure on the premises, built usually

of masonry, which is open to a chimney, as well as to the various closets in the building. The chimney is supposed to maintain a current of air out of the rooms in which the closets are located; this dries the material, which is then burned at intervals.

Where sewers have not been available, crematory systems have been installed in many schools and other public buildings in the United States; but, while sometimes fairly satisfactory for a while, they are usually soon found to be troublesome, expensive, and dangerous. The air-currents sometimes reverse into instead of out of the rooms containing the closets; danger ensues unless the burning is regularly attended to; and, without constant care in the attendance, the whole apparatus is likely to get out of order. Moreover, it is entirely unadapted to the disposal of liquid wastes such as those from sinks, washbowls, laundry basins, and bathtubs, which are as necessary to be taken care of as fæcal matter and urine.

In the foregoing paragraphs (Arts. 4 to 9), various makeshifts for caring for sewage have been described which are not worthy the name of "systems," although the privy vault and the cesspool are in very wide use. We next come to the only methods for removing sewage which are at present worthy of serious consideration when planning a sewerage system.

10. Water-Carriage Systems. Water-carriage systems of sewerage are those in which water is added to the fæcal matter and other foul wastes in such quantities as to permit of their rapid removal by gravity in sewers. As already stated, the water so added usually constitutes 99.8 per cent or more of the resulting sewage.

Water-carriage systems are now so universally used for sewerage purposes, that usually the two terms may be considered synonymous. That is, in the present day, a sewerage system is practically always a water-carriage system.

There are two kinds of water-carriage systems—namely, the *Combined System* and the *Separate System.*

11. Combined System. The combined system of sewerage is that in which the storm sewage flows in the same sewers with the sanitary and the manufacturing sewage. The combined system came into use prior to the separate.

12. Separate System. The separate system of sewerage is that

in which separate sewers are provided for the storm sewage and for the sanitary and manufacturing sewage.

13. Comparative Merits of Combined and Separate Systems. The separate system came into prominence about 1880. At that time and for many years following, there was an active discussion over the relative merits of the two systems, some prominent engineers advocating one, and some the other. At the present time, the discussion has died down, and sanitary engineers use both, adopting whichever is best suited to local conditions, and often using a combination of the two.

In favor of the separate system, the following points have been cited:

1. The sanitary sewage which constitutes the dry-weather flow of combined sewers is so very small in comparison with the storm sewage, that in circular sewers, which are the most economical to build, it forms merely a trickling stream, with little velocity, over the bottom of the large sewers required; while in the separate system the sewers are proportioned for this small volume, and the sewage consequently has good depth and velocity. Moreover, sanitary sewers are free from the sand and other street detritus which are inevitably washed into combined sewers during storms, and which are especially troublesome in forming deposits. Hence, in the separate system, it is easier to make sewers self-cleansing from deposits.

2. Above the low-water line in combined sewers, the extensive interior surfaces of the large sewers required become smeared with filth in times of flood, which remains to decay and produce foul gases after the flood subsides.

3. On account of the comparatively small size of the sanitary sewers of the separate system, it is easier to flush them so as to keep them clean. Automatic flush-tanks can be used at small expense to do this very satisfactorily.

4. On account of the comparatively small size of the sanitary sewers of the separate system, the air in them is much more frequently and completely changed by the daily fluctuations in the depth of sewage and by the currents of air through ordinary ventilation openings. Hence, in the separate system, ventilation is easier and more perfect.

5. In case the sewage has to be purified, the separate system is more economical, because only the sanitary sewage need be treated, the storm sewage being discharged into nearby natural watercourses.

6. In small cities, and in large portions of large cities, the storm water can usually be carried some distance in the gutters, and then removed by comparatively short lengths of storm sewers, laid at shallow depths and discharging into the nearest suitable natural watercourses. In such cases, a separate system of sewers will usually cost only a fraction, frequently only one-third, as much as a combined system. For small towns, the great cost of a combined system would often prohibit the construction of sewers entirely, or postpone it almost indefinitely, were it not that a separate system can be built so cheaply. On this account alone, the introduction of the separate system of sewers has been of incalculable benefit in America.

7. On account of their relatively small size, sewers of the separate system can be made almost entirely of vitrified sewer-pipe, which has the important advantages over brick sewers, of greater smoothness, of being impervious, of having few joints, and of ease in making the joints practically water-tight. It is impossible to make even a pipe sewer absolutely water-tight, and with brick sewers the difficulty is very much greater.

In favor of the combined system, the following allegations, corresponding to the above points, have been made:

1. By making combined sewers egg-shaped with the small end down, or by making a small, semicircular channel in the bottom (see Figs. 19, 24, and 25), the depth and velocity of the dry-weather flow can be made sufficient to cause the sewer to be self-cleansing.

2. The coating on the interior surface of large sewers above the low-water line is not dangerous, and in fact is of very little importance.

3. While it is true that the smaller, separate sewers can be flushed more perfectly for the same expense, the larger, combined sewers are more convenient for removing obstructions, and are flushed out very completely (though at too long intervals in dry weather) by the floods of storm sewage during rains.

4. In regard to ventilation, the larger volume of air over the sewage in the larger, combined sewers dilutes to a much greater degree the gases from the sewage.

5. In case the sewage must be purified, it must be remembered that the early flow of storm sewage from the streets is foul, to some extent, from the droppings of animals and other surface filth; and it may in some cases be questionable whether this may not require purification in addition to the sanitary sewage.

6. Wherever, as in the case of the business districts of large cities, it is necessary to provide as great a length of storm sewers as of sanitary sewers, it will be cheaper to build one set of sewers, as in the combined system, rather than two, as would be required in such districts with the separate system.

The *general conclusions of sanitary engineers* at present regarding the relative merits of the separate and combined systems, are as follows:

a. Either system can be made satisfactory from a sanitary point of view.

b. The cost of a properly designed system, including means for safe disposal of sewage, should ordinarily decide which of the two systems should be built.

c. On the basis of cost, the separate system is usually the better for small cities, for suburban and sometimes residence districts of large cities, and for all cases, even those of large cities, where the sanitary sewage requires treatment while the storm sewage can be safely discharged into nearby watercourses. The separate system has just been recommended for the city of Baltimore on this last account.

d. Similarly, on the basis of cost, the combined system is usually the best for the business and other very thickly built-up districts of large cities, and, in general, where storm sewers must be coextensive with sanitary sewers; also for cases where both storm sewage and sanitary sewage require purification.

e. Often a combination of the two systems can be made to advantage, storm water being admitted to the sewers only in certain portions of the system, such as the business districts.

GENERAL FEATURES OF SEWERS

14. Kinds of Sewers. *Sanitary sewers* are those constructed to carry foul waste liquids of human or animal origin—that is, sanitary sewage. Since sewage of human or animal origin is most apt to contain the germs of human diseases, sanitary sewers require special

precautions in design, construction, and maintenance, to render them safe. Manufacturing sewage is often, however, even stronger and more offensive than sanitary sewage, and hence requires equal precautions. In the separate system, the manufacturing sewage should go into the sanitary sewers or into special sewers of similar character.

Combined sewers are those constructed to carry both sanitary sewage and storm sewage. With the combined system, the manufacturing sewage also usually goes into the combined sewers.

Storm sewers are those constructed to carry storm sewage only.

An *outlet sewer* is one connecting a sewer system, or a part thereof, with the point of final discharge of the sewage.

A *main sewer*, or *sewer main*, is the principal sewer of a city, or of a large district thereof, into which branch sewers discharge.

A *sub-main sewer* is a branch of a main sewer, receiving in its turn the discharge of smaller branches.

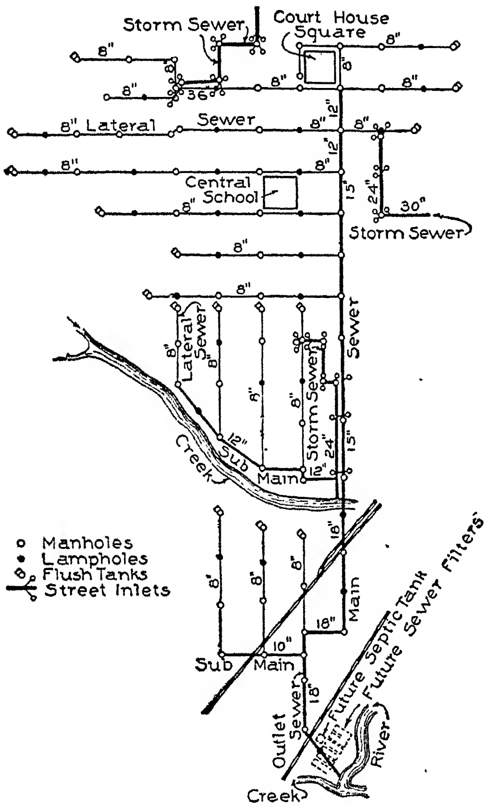


Fig. 4. Kinds of Sewers and Arrangement of Accessories.

A *lateral sewer* is one not receiving the discharge of other sewers, hence serving only property closely adjacent.

In Fig. 4, the various kinds of sewers above described are shown, from a portion of the actual sewerage map of a small city, sewered on the separate system.

15. *Intercepting sewers* are those built across lines of other

sewers, to intercept the sewage flowing in them and carry it away to different outlets.

In Fig. 5 are shown the intercepting sewers of the city of Chicago, built along the lake front to intercept the sewage in the sewers which formerly discharged into and polluted Lake Michigan, from which the water supply of the city is taken. From the intercepting sewers, the sewage is pumped into the Chicago River, which now discharges through the great Drainage Canal into the Desplaines river, the Illinois River, the Mississippi River, and the Gulf of Mexico.

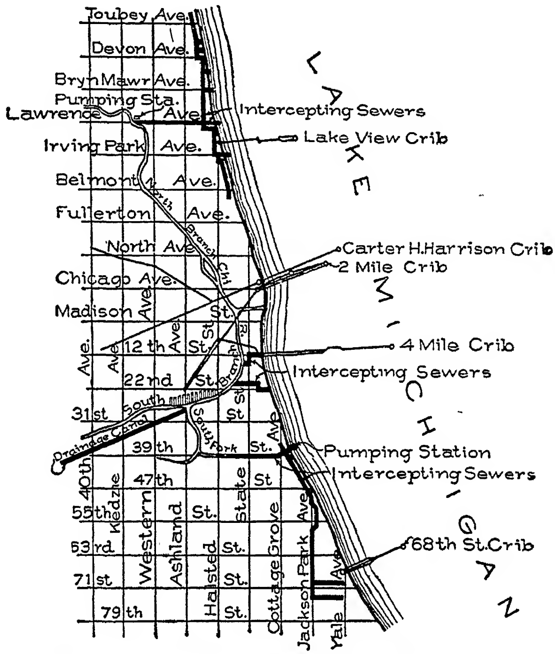


Fig. 5. Intercepting Sewers of the City of Chicago, Ill.

16. General Description of Sewers. Sewers, as usually built, are smooth pipe or masonry conduits, as nearly water-tight as practicable, buried in the ground as deeply as necessary to serve the adjacent houses and drain other territory tributary upstream. They are very carefully constructed to an exact grade line, determined by the engineer who made the sewer plans.

Unless special circumstances require other forms, sewers are usually made circular, this shape giving the greatest strength and area for a given amount of material. For other shapes, and the circumstances to which they are adapted, see Figs. 19 to 25.

The *invert* of a sewer is the lowest point on the interior surface (being so called because the interior curve is there inverted). When the grade of a sewer is mentioned, or the elevation of the sewer at a

given place is spoken of, the invert is always meant. The invert is also sometimes called the *flow line.*

Almost all sewers up to 24 inches' diameter, and many from 24 to 36 inches' diameter, are made of vitrified or cement pipe. Above these sizes, concrete or brick masonry is ordinarily used. Stone masonry and iron pipe are also used, but only seldom. A comparison of these materials is given elsewhere in this paper.

At intervals along sewers, *manholes* (Art. 21) and *lampholes* (Art. 22) are placed to permit examination and repairs, and often *flush-tanks* (Art. 23) are provided to keep the sewers clean. In the case of storm sewers and combined sewers, either *street inlets* or *catch-*

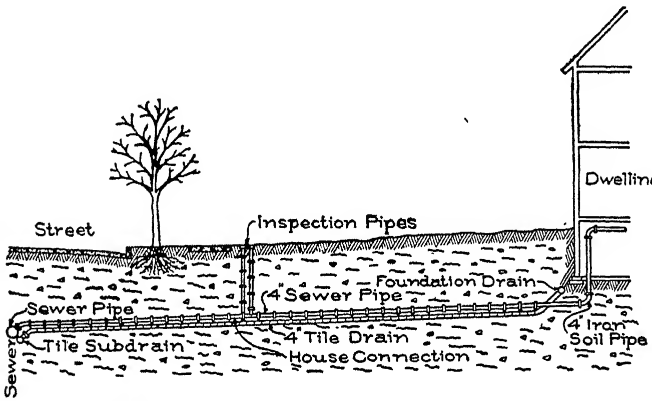


Fig. 6. Street Sewer, Subdrain, and House Connection.

basins (Art. 27) must be provided, for admitting the storm water to the sewers. These are usually placed at or near the curb corners at the street intersections.

A general idea of the relation of a sewer to a building served by it, may be gained from Fig. 6. The sewer there shown is a pipe sewer. Usually all lateral sewers are made of pipe; and in the separate system, the submains and mains also, unless the city is quite large.

17. Location of Sewers. Sanitary sewers are usually placed on the center lines of the streets, so as to give equal fall from the houses on both sides. On this account, water, gas, and heating mains, storm sewers, and other conduits should be constructed far enough from the center lines not to interfere with the sanitary sewers. Not

infrequently the center of the street is found already occupied by other conduits which were located without proper foresight; and it is then necessary to place the sewer nearer to one side than the other.

In cases of streets on side hills, it is sometimes necessary to place the sewer close to the downhill side of the street, in order to serve houses on that side which are lower than the street grades.

In a few cases of excessively wide avenues, especially if paved, it is cheaper to build two lines of sanitary sewers, one on each side, than to construct the longer house connections required.

In any town having a fairly extensive system of alleys, careful consideration should be given by the sewerage engineer to the feasibility and desirability of locating part or all of the sanitary sewers in them instead of in the street. In Memphis, this plan was followed as far as practicable. It is not usually feasible to locate combined or storm sewers in alleys, because such sewers must receive storm water from the streets running in both directions, and hence must usually have the street inlets placed at the street corners.

Streets vs. Alleys for Sanitary Sewers. Location of the sanitary sewers in the alleys has a great advantage in avoiding the tearing up of the streets and pavements for sewer repairs and for new house connections, which not infrequently causes them serious injury. Pavements are often ruined by the trenches dug for water, sewer, gas, and other connections. Also, if the sewers are in the alleys, the trenches for house connections do not cross the lawns in front of the houses.

On the other hand, the system of alleys in the ordinary town is a public nuisance. They are usually filled mainly with manure piles, garbage, and debris of all descriptions; and they open through the middle of the blocks vistas which suggest most forcibly a neglected city dumping ground. Owing to their vile sanitary condition, the alleys are usually the first danger spots demanding attention when a town is threatened with an epidemic. Except in the business districts where they can be paved and policed, there is no necessity for alleys unless the lots are very narrow, for in almost every town there are sections which do without and never miss them. Teams can without inconvenience drive in from the front, along a cinder or gravel drive. Such sections are better off without the alleys, from both the sanitary and the æsthetic points of view.

For the above reasons, it is often unwise to perpetuate, or perhaps even extend, the alley system by locating sewers in them.

Again, the system of alleys, more often than not, is far from being as complete as the street system; and in such cases it will usually add considerably to the total length of sewers required to serve a given territory, if part of them are placed in the alleys. The alleys, also, are usually too narrow to permit the construction of sewers of considerable depth, without trouble as regards the excavated material, the handling of pipe, etc. Moreover, houses and the fixtures in them are usually so located that the house connection would be longer to the alley than to the street, requiring a deeper sewer for equal service. This, however, is not always the case.

The sanitary engineer should study each town by itself, and decide this question after giving due weight to all these various considerations.

18. Depth of Sewers: The depth of sanitary and combined sewers should be great enough to afford good drainage to the basements of all buildings. This will usually call for the tops of the sewers to be about $3\frac{1}{2}$ feet below the basement floors, as follows:

MINIMUM DEPTHS FOR SANITARY AND COMBINED SEWERS

Fall from sewer to house	2 ft. 0 in.
Fall from basement floor to house connection	1 ft. 6 in.
Total from *top* of sewer to basement floor	3 ft. 6 in.
For *sewer laterals*, add to the above for fall at sewer	1 ft. 0 in.
Total from *invert of lateral sewer* to basement floor	4 ft. 6 in.
For *residence districts*, add for ordinary-depth basements below street level	4 ft. 0 in.
Total minimum depth to invert of *lateral sewers in residence districts*	8 ft. 6 in.
For business districts, add for ordinary-depth basements	8 ft. 0 in.
Total minimum depth to invert of *lateral sewers in business districts*	12 ft. 6 in.

Hence, under average conditions, the depth of sanitary and combined pipe sewers of 12-inch diameter and less, should be not less than $8\frac{1}{2}$ feet in residence districts, and $12\frac{1}{2}$ feet in business districts. If, however, there is only a short stretch of low-lying ground on a residence street, it may be advisable to reduce the above depth, say to 6 feet as a minimun, when by so doing a very long stretch of sewer can be lessened that much in depth throughout, and a large saving in cost made thereby.

In the case of sanitary and combined sewers more than 12 inches in height, the above depths should be increased by the excess over 12 inches, for the house connections should enter near the top of the sewer.

In the case of storm sewers and of outlet and intercepting sewers, the depth will no longer be determined by the depth of basements alongside. In these sewers three other considerations determine the depth: (1) the depth at the upper end necessary to afford a good outlet for the sewage; (2) the grade necessary to give good velocity; (3) the depth necessary to prevent injurious heaving of the sewer foundations by frost.

In regard to the third point, no danger need be apprehended of the sewer itself freezing up, even if it be laid practically at the surface, for a stream of warm, flowing sewage will not freeze. There will be little or no danger of trouble from heaving, if the sewer foundation be four feet under ground; and many stretches of pipe sewers only two or three feet deep operate with entire satisfaction even in the northern United States.

19. Subdrains. It has already been stated that sewers should be made as nearly water-tight as possible. Otherwise there would be danger of the sewage leaking out so as to contaminate the adjacent soil. Hence, while it is not possible at any reasonable expense to make sewers *absolutely* tight, they should be built with the utmost care in this particular.

Yet, when due care is used in this respect, the sewer is made unfit for performing another important duty—that of draining away subsoil water so as to dry out unwholesome dampness from the soil, and especially from wet cellars and from under and around houses built on low ground.

In order to secure such drainage, and also, in case of wet ditches, to help remove water from the trenches during construction, it often becomes necessary or advisable to add to the sewer a *subdrain.*

A *subdrain* is a line of drain tile or sewer pipe laid with open joints, in the same trench with the sewer.

To allow connections with cellar drains to be made from both sides of the streets, the subdrain should be placed with its top a few inches below the bottom of the sewer; and to leave a firm foundation

for the sewer itself, the subdrain should be placed a little to one side of the sewer.

With the above arrangement, special care should be taken to make the sewer joints tight, and there is some danger of slight leakage of sewage into the subdrain. Such leaks tend to stop themselves as time passes.

It is not safe to connect cellar drains directly with a sewer, even though they are trapped to prevent the sewer air from penetrating into and filling the pores of the soil under houses. In dry times, there may be no water running in the cellar drains; and at such times the water in a trap may evaporate so as to unseal it. Cellar and foundation drains should be connected to the subdrain instead of to the sewer itself.

The general relation of the subdrain to the sewer in the street, and the method of connecting it with the foundation drains, may be seen in Fig. 6.

In construction, the joints of the subdrain should usually be wrapped with muslin to prevent the entrance of mud and sand. The cloth, of course, does not last long; but by the time it rots, the soil around the tile will usually have become recompacted so that there is no longer danger of its getting into the drain. In quicksand, it may sometimes be necessary to fill in fine pebbles or broken stone around the subdrain.

20. House Connections. In Fig. 6 is also shown the method of connecting the sewer itself with the iron soil-pipe which drains the different plumbing fixtures, and which should extend at least 6 feet outside the basement wall. The house connection should be a line of 4-inch vitrified sewer-pipe, laid at right angles to the sewer, with tightly cemented joints, and if possible to at least a 2 per cent grade (that is, with a fall of 2 feet in 100 feet length). Some prefer 6-inch house connections; but these should not be allowed with 8-inch sewers, as the house connection may then allow obstructions to be carried to the street sewer large enough to catch therein and cause stoppages. At the sewer, the house connection should turn down, by a 4-inch 45-degree elbow, into a 4-inch Y-junction laid so as to slant upward 45 degrees—all as shown in Fig. 7. This slant upward

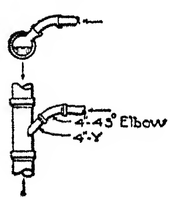


Fig. 7. Junction of House Connection with Sewer.

keeps the Y from affecting the smooth ordinary flow in the sewer.

In case the sewer is more than 12 feet deep below the street surface, the expense of digging down to it in making house connections would be so great that it is usually better, while the trench is open during sewer construction, to put in a *deep-cut house connection*, as shown in Fig. 8. In this case, sewer pipe must be used from the subdrain also, if such a drain is used; and care should be taken to turn the bells of the subdrain connection down so that the plumbers need make no mistake in the connections afterwards.

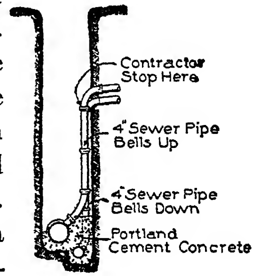


Fig. 8. Deep-Cut House Connection.

In sewer construction, a Y-junction for a house connection (or a deep-cut house connection, if the sewer is over 12 feet deep), should be conveniently located opposite each lot on each side of the sewer; and the ends should be stopped with vitrified stoppers, covered over with sand and then cemented in. Full and accurate records must be kept of the exact locations of these connections, so that they can be found without trouble at any time.

No person should be allowed to cut or break into a pipe sewer for making house connections or any other kind of junction. If there is no Y or T-branch already set for the connection, a full length of pipe should be broken out and the proper Y or T-branch inserted. A skilful workman can readily do this by breaking off one-half the bell of the new pipe, and of that of the old piece into which it must be inserted, and turning the new piece half around after insertion. The joints must then be re-cemented with great care.

21. Manholes. It has already been stated (Art. 16) that manholes must be placed at intervals along sewers, to permit of examination and repairs. These manholes are usually circular brick wells, with Portland cement concrete bottoms and heavy cast-iron covers, as shown in detail in Fig. 9. They must be large enough at the bottom, and for a couple of feet above the top of a pipe sewer, to permit a man to work comfortably. Four feet in diameter is a satisfactory size. Sometimes the manholes are made elliptical at the bottom, with the long axis lengthwise of the sewer; but this form is more difficult to build. Above the point mentioned, the sewer may be drawn in gradually to a diameter of about 2 feet 9 inches, at a point

2 feet 9 inches below the street surface, and thence narrowed more rapidly to about 20 inches diameter at the bottom of the cover casting.

The cover casting may be of any manufacturer's design satisfactory to the engineer, weighing at least 375 lbs. The lid should usually be perforated with 1-inch holes, to permit ventilation of the sewer; and immediately below it, there should be hung a heavy cast-iron *dustpan*, to catch any dirt entering through the perforations.

There should be a ladder of iron rungs built into the walls, as shown in Fig. 9.

The channels in the concrete bottom should be very carefully formed to give smooth, true, circular channels. They are sometimes lined with split sewer pipe. The benches at the sides of the channels should slope down towards the channels, as shown in the figure.

The concrete for the bottom may be made of 1 part Portland cement, 3 parts sand, and 5 parts of broken stone. All the brickwork should be laid with tight *shove joints*, in 1-to-3 Portland cement mortar; and the manhole walls should be plastered both inside and outside with 1-to-2 Portland cement mortar.

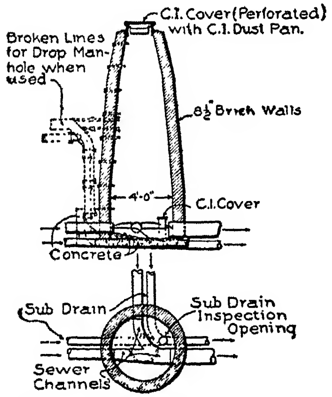


Fig. 9. Sectional Elevation and Plan of Sewer Manhole.

Should sudden drops in the sewer be desirable, they can be made at *drop manholes*, in the manner shown by the broken lines of Fig. 9.

In the case of large masonry sewers, which often are many feet in diameter the manholes may be joined directly to the masonry of the upper part of the sewer.

Opinions of sanitary engineers differ somewhat as to the distance apart at which manholes should be placed. In general, a manhole should be placed at all junctions of sewers, and at every change of grade or alignment in all sewers but those large enough to be entered readily for cleaning. This means that sewers should ordinarily be perfectly straight between manholes, to facilitate inspection and repairs, all changes in both grade and alignment being made at the manholes themselves

Also, in any part of the system—such as in the business district—where it is especially objectionable to have the street dug up for repairs, manholes should be placed at least as often as every city block—that is, 300 to 400 feet apart. In the other parts of the system, some engineers leave out every other manhole where the grade and alignment are straight, putting manholes at least every two blocks. The intermediate manholes left out are replaced by *lampholes* (Art. 22) to save cost. In Figs. 4 and 38, the above arrangement of manholes is shown in two actual sewer systems.

22. Lampholes. The lampholes which, to save cost, are sometimes adopted in place of part of the manholes, consist each of a vertical line of sewer pipe, with cemented joints, reaching to the street surface, as in Fig. 10. Usually 8 inches is the minimum diameter for this pipe, which is cemented at the bottom into a regular sewer-pipe T-junction. Some concrete should be placed under and around this tee for a foundation. At the street surface, there should be an iron casting similar to a manhole casting, but smaller, as shown in Fig. 10.

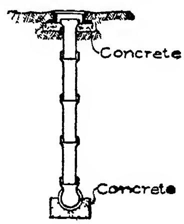


Fig. 10. Lamphole.

The earth, in refilling, needs to be very thoroughly tamped around the lamphole; and the lamphole casting should not be set until the material is thoroughly settled.

The object of the lamphole is to permit of inspection of the sewer, in determining whether it is clean and in locating stoppages. While its name suggests the lowering into it of a lamp, a beam of sunlight reflected into it from a mirror is more convenient.

A lamphole usually costs about $30 to $35 less than a manhole.

In Figs. 4 and 38 the above arrangement of lampholes in two actual sewer systems may be seen.

23. Flush-Tanks. Near the upper ends of sewers the flow of sewage is very small, sufficient only to make a shallow, trickling stream, liable not to be able to carry along the solid matter in the sewage so as to prevent deposits. An 8-inch lateral sewer in a residence district in a small town, even if laid at the minimum grade, would usually have an average depth of flow in the upper two and one-half blocks of less than one inch. Hence it is desirable, though not always absolutely necessary, to provide some special means for

regularly flushing the upper portions of sewer laterals, to make them self-cleansing.

Again, in low-lying, level districts, it may be necessary, on account of the lack of fall, to lay the sewers at such slight grades that the velocity is insufficient to prevent deposits. Here, too, some special means should be provided for regularly flushing the sewers.

In the case of pipe sewers, such as are ordinarily used for the laterals in all systems, and for most of the mains in separate systems,

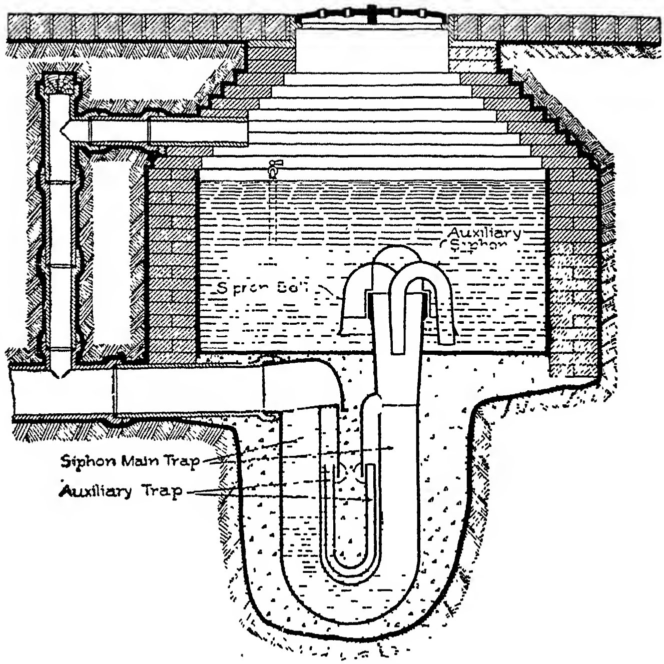


FIG. 11. Sewer Flush-Tank with "De La Hunt" Adjustable Siphon.

the most efficient and reliable means for securing regular flushing is the use of automatic flush-tanks.

A *flush-tank* is a masonry cistern built in the street, above the grade of the sewer, filled by a constantly running stream of water brought by a small pipe from the water-supply mains, and suddenly emptied by automatic devices into the sewer whenever the high-water line is reached.

Flush-tanks usually have a capacity of 150 to 500 gallons, and should approach the larger size named, to secure an efficient flush

for two or three blocks. When made separate from manholes, flush-tanks are usually circular and of the general design of the masonry tank shown in Fig. 11. It is usually better, however, to combine the flush-tank with a manhole, as is shown by the masonry tank and manhole in Fig. 12. This permits inspection of the flush-tank and sewer, and is cheaper than to build manhole and flush-tank separate.

The bottoms of flush-tanks are usually of Portland cement concrete, and the walls of brick laid in Portland cement mortar. The

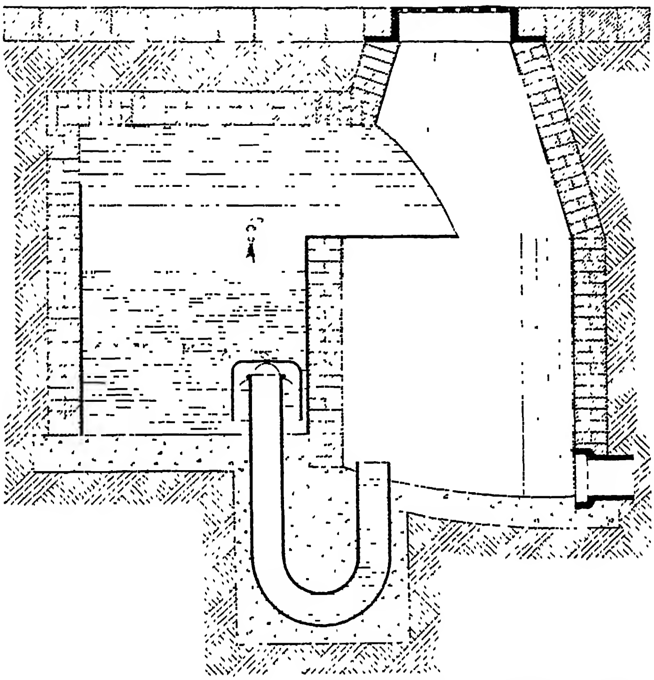


Fig. 12. Combined Flush-Tank and Manhole with Special "Miller" Siphon.

tanks should be plastered inside and outside as described for manholes (see Art. 21). Special care should be used to make flush-tanks absolutely water-tight.

The water is usually brought to the flush-tank by a $\frac{3}{4}$-inch galvanized pipe from the nearest water main. This pipe must be laid below the frost line ($5\frac{1}{2}$ to 7 feet deep, in the northern part of the United States), but should be turned up after it enters the flush-tank so as to discharge above the high-water line, as shown in Fig. 11.

The flush-tank may be prevented from freezing by being connected with the sewer above the high-water line, as shown in Figs. 11 and 12, so as to admit the warm air from the sewer.

It is a quite common practice to place flush-tanks at the heads of all laterals, as illustrated in Figs. 4 and 38. While some engineers dispute the necessity for this, it must be admitted that such an arrangement will be of great benefit, and its adoption is here advised for most cases.

In Fig. 38 the use of flush-tanks is shown at certain half-way points on the long laterals. The necessity for this arose from the fact that the sewers were not to be completed to the north ends of the laterals for some years after the southern portions were built.

The writer of this paper has used flush-tanks with success and great benefit, at intervals of about two or three blocks on sewers laid at grades below those considered necessary to make the sewers self-cleansing, though part of the flush from the intermediate tanks flows some distance upstream at each discharge.

The flush-tanks of a sewer system should be frequently inspected after the sewers are put into operation, and should be carefully kept in working order. The things needing most faithful watching are: first, the automatic discharging apparatus; and, second, the supply of water. The faucet admitting water may readily become choked up, putting the flush-tank out of service, or, on the other hand, may get wide open, wasting thousands of gallons of water every day.

24. Automatic Flushing Siphons. The reliability of flush-tanks in actual use will depend upon the frequency and care with which they are inspected and kept in working order, and especially on the reliability of the automatic discharging apparatus. No discharging apparatus having moving parts should be used in flush-tanks. Such apparatus is too likely to get out of order.

In Figs. 11 and 12, *sewer siphons* are shown for automatically discharging the flush-tanks suddenly whenever they fill to the high-water line. Such siphons have no moving parts whatever to get out of order, and should always be employed with flush-tanks.

In Fig. 11 the four ordinary parts of a flushing siphon are indicated. All four are usually iron castings, and must be air-tight. The *siphon bell* rests upon the *main trap,* which latter, together with the *auxiliary trap,* must be filled with water to the heights of the

short legs, before the bell is placed in position. The main trap must be set plumb. The *auxiliary siphon* serves to ensure, at the end of the discharge, the *venting* of the siphon—that is, the free admission of air to the inside of the bell. With clear water, the auxiliary siphon is not always used; but it should be used whenever the siphon is to be used with raw sewage.

In the working of the siphon, the water in the flush-tank confines the air inside the bell and above the water in the main and auxiliary traps, and puts it under increasing pressure as the water rises. When the high-water line in the flush-tank is reached, this pressure becomes so great that the water in the auxiliary trap is forced down to the very bottom of the trap, and the confined air then blows out of the short leg of the auxiliary trap, thus releasing the air-pressure inside the bell, which up to this time has held back the water in the flush-tank. The water in the flush-tank then rushes out into the sewer through the main trap, and by siphonic action will continue to flow out until drawn down to the level of the bottom of the bell. Air then enters the bell through a small *sniff-hole* provided near the bottom of the bell for this purpose, *breaking* the siphonic action—that is, *venting* the siphon.

In case a siphon is used for raw sewage, there is often difficulty in securing satisfactory venting of the siphon at the close of the discharge; but this trouble can be remedied by using an *auxiliary siphon,* as shown in Fig. 11, and as illustrated by broken lines for the "Miller" siphon in Fig. 12.

In the *Miller siphon,* shown in Fig. 12, there is no auxiliary trap; but at high-water line the air-pressure in the main trap becomes so great that a bubble escapes, taking with it enough water from the short leg to start a sudden rush of water from the tank into the main trap, which suffices to establish siphonic action. This greatly simplifies the siphon; and the principle can be relied upon for siphons not larger than about eight inches internal diameter of the main trap. Larger siphons should have auxiliary traps.

In some siphons—as, for example, the *Rhoads-Miller*—the auxiliary trap is cast as a part of the main trap, out of which it opens below the floor of the tank, being entirely buried out of sight and reach in concrete. An objection to auxiliary traps such as shown in Fig. 11, is that they are inaccessible and may in time become

stopped up. However, they make the action of large siphons more certain.

25. Hand-Flushing of Sewers. For large sewers, flush-tanks and siphons would have to be extremely large to be effective. Even in small sewers the effect of the flush will not be great for many blocks below the tank. Some engineers doubt the necessity for very extensive use of flush-tanks. When flush-tanks are not properly inspected and regulated (as to the feed faucet), they sometimes waste great quantities of city water. For these reasons, and sometimes to save cost, hand methods are sometimes relied upon for flushing sewers.

The most convenient, economical, and effective hand-flushing device is a connection with a water main by a water pipe of size large enough to flush the sewer very thoroughly. The only labor then required is that necessary for opening and closing the valves on this pipe. Such a flush, continuing much longer than the discharge of a flush-tank, can be made effective through a long stretch of sewer. The objections are the trouble and the danger of neglect inherent in hand work, and the usual greater length of time between flushings. To flush the sewers daily would be very expensive, both as to labor and as to the large amount of water needed.

Occasionally, very favorable local circumstances may permit of the admission at will of large volumes of water for flushing purposes from a stream or lake higher than the sewer.

In some cases, hand-flushing is done by temporarily damming up the sewage itself, and then suddenly releasing it when sufficient head has been secured.

A fire hose run to a manhole from a nearby hydrant may be the resort in other cases. In extreme cases, water has even been hauled to the sewer in tanks, for flushing.

26. Sewer Ventilation. More fear used to be felt of the danger of *sewer gas* (more properly termed *sewer air*, see Art. 1) in communicating disease, than medical knowledge warrants at the present time. Nevertheless, it is very important, not only from the sanitary but from many other points of view, that sewer air should be as pure as possible; and this requires good ventilation of the sewers. Fresh-air currents in the sewers should be maintained in some reliable way.

One method of securing this is to use perforated manhole covers (see Fig. 9). Objection is sometimes made to these as letting objec-

tionable odors out into the street; but with well-designed and well-constructed sewers, well flushed and well ventilated, there will be no cause for complaint. If there are seriously objectionable odors from the manholes, such odors should be considered valuable as notices that the sewers are in dangerous condition, demanding immediate work to make them safe. Sewer air escaping into streets through manhole-cover perforations, is at once so diluted by fresh air as not to be dangerous to the health of passers by.

Another effective means for securing good ventilation is to extend the cast-iron soil-pipes (which form the main drainage pipes in the plumbing systems of houses) untrapped and full size through the roof. Figs. 4 and 35 show the omission of traps on the soil pipe. In Fig. 35, however, the use of a *disconnecting trap,* to disconnect the sewer air from that in the house plumbing pipes, is shown by broken lines. In case this is used, a ventilating pipe for the sewer should be extended up the sides of the house from the sewer side of the trap, and a fresh-air inlet provided on the house side, both as shown by the broken lines in Fig. 35.

The use of perforated manhole covers and untrapped soil pipes extending through the roofs, is all that is required to secure good ventilation of the sewers, the house connections, and the soil pipes themselves. Their use provides a large number of openings at different levels; and the temperature of the air in the sewers is practically always different from that above the ground. Hence air-currents are maintained for the same reason that chimneys cause draughts for fires, and a good circulation of air is maintained.

In the past, experiments in sewer ventilation have been made with tall chimneys, fan blowers, etc.; but such devices are entirely unnecessary, are very costly, and are usually unsuccessful on account of the very large number of openings into the sewer, which limit the air-currents produced by such devices to short distances.

27. Street Inlets and Catch-Basins. In the case of storm sewers and combined sewers, means must be provided for admitting the storm water to the sewers from the streets. For this purpose, either *street inlets,* as shown in Fig. 13, or *catch-basins,* as shown in Fig. 14, may be used. If the water can be allowed to flow one block safely in the surface gutters, the inlets for storm water would need to be only at each street intersection. In a few cases they need to be

closer; but in many more cases the storm water can be carried in the gutters for two or even a greater number of blocks without injury, thus greatly reducing the number and cost of storm sewers and of inlets for storm water.

The simplest and least expensive arrangement for admitting storm water is the *street inlet,* which, as shown in Fig. 13, is a mere branch sewer, with a grated opening from the street. Besides costing less, the street inlet is often preferred for sanitary reasons, as it does not retain foul, unsanitary deposits, as does the catch-basin.

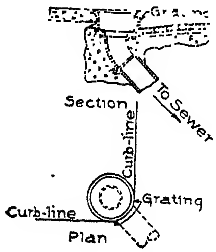


Fig. 13. Street Inlet.

The *catch-basin,* shown in Fig. 14, is designed to catch the sand, dirt, and other heavy street detritus, and prevent their entering the sewer. Unless catch-basins are frequently cleaned, however (which is very seldom the case), they fail almost entirely in this; and as they are usually well filled with more or less foul deposits, they are condemned by many engineers.

When street inlets and catch-basins are left untrapped, as shown in Figs. 13 and 14, they assist in the ventilation of the sewers. This is sometimes objected to on account of the opportunity for the escape of foul odors, and traps are introduced in both, as shown by the dotted lines in Fig. 14, to prevent ventilation of the sewers through the storm inlets. If the sewers are kept in as good condition as they should be, there will be no good ground for such objections.

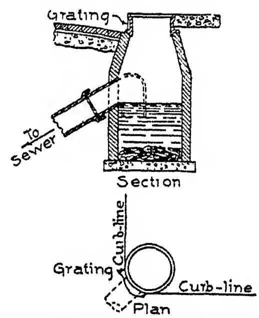


Fig. 14. Catch-Basin.

28. Inverted Siphons. It sometimes becomes necessary or desirable to carry a sewer down below the regular grade line, to pass under some obstacle or depression, and to raise it again to the regular grade line beyond. Such a stretch of sewer will necessarily flow full and be under some pressure. It is called an *inverted siphon.* The necessity for the use of the inverted siphon may be occasioned by some stream, by railway tracks, by another sewer, by a large water main, or sometimes merely by a low stretch of ground which happens to lie at such a level

that the sewer cannot be carried across it at the regular grade.

Inverted siphons have often been constructed and operated successfully. It is wise, however, to take certain precautions in their design and construction, as otherwise serious trouble may be experienced with them.

First, as to material, it may be said that ordinary sewer pipe is not well suited to carry sewage under pressure, on account of the great difficulty in making absolutely tight joints, and on account of the brittle and unreliable nature of the pipe as to resistance to bursting pressures. If used under pressure, pipe sewers should be subjected to only a few feet of head, and all joints should be thoroughly encased in impervious Portland cement mortar and concrete, reinforced with imbedded steel bands. Brick masonry is still less suited to with-

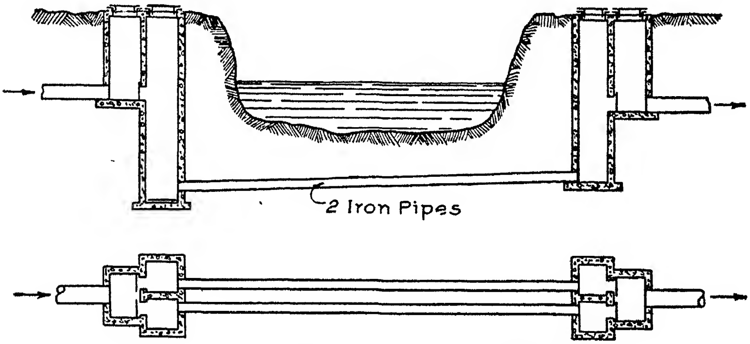


Fig. 15. Sectional Elevation and Plan of Inverted Siphon.

stand bursting pressures. Ordinarily iron pipe should be used for inverted siphons.

Second, it is especially important to insure a current in the inverted siphon sufficiently rapid to prevent deposits. If the flow is light at first, to increase afterwards, as is often the case, it is well to divide the siphon into two or more pipes with valves on each, so that the entire flow can be turned into one at first. If it is easy to add the second pipe in the future, it may often be left out at first. Thus in Fig. 38, the inverted siphon from the 18-inch outlet sewer to the septic tank is at present only an 8-inch cast-iron pipe, with provision for adding a 12-inch cast-iron pipe later.

Third, the design should be such as to permit ready access for inspection and removal of obstructions. The inverted siphon should,

if possible, be so planned that the flow of sewage can be diverted for a short time, either into one pipe, or entirely away from the siphon; and the siphon should drain to a low point from which the contents can be removed by gravity through a blow-off or by being pumped out. Where feasible, and especially where it will be very difficult (as under a stream) to dig down to the siphon in emergencies, the siphon should be made absolutely straight in grade and alignment, and a manhole placed at each end.

In Fig. 15 is shown an outline of an inverted siphon designed according to the above principles.

Where the siphon can readily be opened for repairs, as is the case with the one in Fig. 38, such expensive construction need not be resorted to. The one in Fig. 38, which carries sewage across low ground to a sewage tank about seven feet above the surface, is laid at an average depth of about six feet, and neither the grade nor the alignment is straight. It drains, however, to a low point, where a blow-off into a sewer is placed.

29. Outlets for Sewer Systems. We have heretofore discussed the house connection, and the laterals, submains, and main sewers, with their manholes, flush-tanks, and other accessories. We come next to the *outlet*, which, though not considered first here, would be one of the first things a sewerage engineer would have to consider in designing a sewer system.

Where possible, all of the sanitary sewage or combined sewage of the city should be led to one outlet, as the cost of disposing of it properly may be lightened thereby, and as the danger of injunction suits and other legal difficulties arising from damages from impurified or only partially purified sewage may be multiplied with the number of outlets. Often this will be possible by constructing comparatively short lengths of deep sewers where at first sight the topography would seem to make it impossible to secure one outlet. The size of the city, as well as the topography, will affect the number of outlets.

Storm sewage in the separate system can usually be discharged through a number of outlets into nearby natural watercourses.

Great effort should be made to secure an outlet or outlets for the sewer system low enough to drain all parts of the city by gravity. Pumping of the sewage or a material part of it, will mean a continuous expense involving an amount which would be sufficient to

pay the interest on a large initial expense to secure a gravity outlet. Besides, there is the danger of such apparatus failing at critical times.

Usually effort is made to secure, if possible, an outlet into a considerable stream or body of water, even if the sewage is to be purified.

30. Sewage Disposal. Heretofore, sewage has been disposed of, in the great majority of cases, by simply emptying it into the largest available stream or body of water near at hand. Such serious contamination of natural waters has resulted from this practice, that at the present time much more attention than formerly is being paid to sewage purification; and usually the outlet plans should be made with the expectation that some method of purification will have to be adopted in the future, if not at present.

Sewage disposal is discussed further on, at much greater length (see Arts. 110 to 124). It will only be said here that the methods at present in favor almost all involve passing the sewage through large tanks, and then through some form of filter.

SEWER MATERIALS AND CROSS-SECTIONS

31. Sewer Materials. Sewers 24 inches in diameter and under, are usually built of *vitrified sewer-pipe*. A 24-inch pipe sewer, laid to a fall of 0.2 feet in 100 feet, will carry the sanitary sewage, under average conditions, of 29,000 people; and hence it is evident that in separate systems, all the sanitary sewers will be made of pipe, except a few main and outlet sewers in large cities. Considerable percentages of storm sewer and combined sewer systems will be pipe sewers also.

Occasionally *cement sewer-pipe* is used instead of the vitrified pipe.

Sewers 30 inches and larger in diameter, are most frequently built of *brick*. Pipe is sometimes used, however, for 30-inch to 36-inch sewers.

Concrete has of late years been growing in favor, to take the place of brick in sewer construction.

Stone was formerly used to a considerable extent for sewers; but on account of its roughness, and the great cost of cut-stone masonry, stone is suited only for backing brick linings in larger sewers. Even here, concrete would now ordinarily be employed, as both cheaper and better.

Occasionally, as in the case of submerged-outlet sewers into bodies of water, or sewers across marshes on soft foundations, *wooden stave pipe* is used for sewers. These pipes are made of pieces of timber, usually about two inches by four inches in size, put together breaking joints in the field, and hooped at regular intervals with iron bands which can be screwed tight. Wood should be used only where it will be wet all the time, to prevent rotting.

Cast-iron pipe, such as is used for water mains, is often adopted for short stretches of sewer under railways or streams where great strength is essential; for inverted siphons; and in cases where absolutely water-tight joints are essential, such as submerged lines in lakes, harbors, and stream crossings, or where there is much ground water.

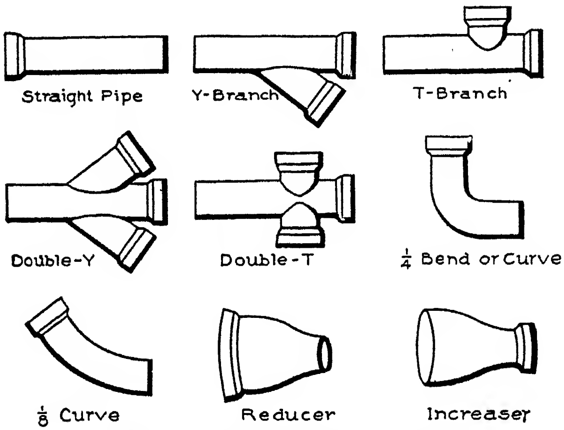


Fig. 16. Vitrified Sewer-Pipe and Specials.

32. Vitrified Sewer-Pipe. Vitrified sewer-pipe has many excellent qualities for sewer use. It is hard, impervious, smooth, strong, does not decay or disintegrate, and is not affected by chemicals. It has few joints as compared with brickwork, and these joints are of convenient shape to make practically water-tight. Vitrified sewer-pipe is readily handled and laid in sewer construction. The materials of which it is made are widely distributed, and hence the cost of the pipe is reasonable.

In Fig. 16 are shown the general forms of the straight pipe and also of the special fittings (*sewer-pipe specials*) most commonly used in sewer construction.

In Table I (page 35) are given standard dimensions for straight sewer-pipe.

Vitrified sewer-pipe is made from shale clays, in very much the same way as brick and other clay products. The temperature at

which it is burned in the kilns must be very high, as in the case of paving brick, so as to produce an "incipient vitrification," a softening and running together of the particles of clay, which gives, on cooling, a very hard, impervious, and strong structure. Smoothness of interior and exterior surfaces is secured by the use of salt during the process of burning, so as to produce a "salt-glazed," glassy skin.

TABLE I

Standard Dimensions for Sewer Pipe

STANDARD				DOUBLE STRENGTH OR EXTRA THICK			
Inside Diam Inches	Thickness of Shell Inches	Depth of Socket. Inches	Weight per Ft. Lbs.	Inside Diam. Inches	Thickness of Shell. Inches	Depth of Socket. Inches	Weight per Ft. Lbs.
8	$\frac{3}{4}$	$2\frac{1}{2}$	22	8	$\frac{7}{8}$	$2\frac{1}{2}$	25
9	$\frac{3}{4}$	$2\frac{1}{2}$	27	9	$\frac{7}{8}$	$2\frac{1}{2}$	30
10	$\frac{7}{8}$	$2\frac{1}{2}$	30	10	1	$2\frac{1}{2}$	34
12	1	$2\frac{1}{2}$	41	12	$1\frac{1}{8}$	3	50
15	$1\frac{1}{8}$	3	60	15	$1\frac{1}{4}$	3	70
18	$1\frac{1}{4}$	3	80	18	$1\frac{1}{2}$	3	100
20	$1\frac{3}{8}$	3	95	20	$1\frac{2}{3}$	3	120
21	$1\frac{3}{8}$	4	105	21	$1\frac{3}{4}$	4	140
24	$1\frac{5}{8}$	4	135	24	2	4	180
27	2	4	215	27	$2\frac{1}{4}$	4	240
30	$2\frac{1}{4}$	4	270	30	$2\frac{1}{2}$	4	300
33	$2\frac{3}{8}$	$4\frac{1}{2}$	320	33	$2\frac{5}{8}$	$4\frac{1}{2}$	340
36	$2\frac{1}{2}$	5	365	36	$2\frac{3}{4}$	5	390

The bells are made large enough to allow an annular space for cement, ranging from $\frac{3}{8}$ inch thick for 8-inch pipe to $\frac{3}{4}$ inch for 36-inch pipe.

Smaller sizes of pipe, down to 3 inches in diameter, are made.

Double-strength pipe is used only in cases requiring unusual strength.

Vitrified sewer-pipe must be carefully inspected, piece by piece, just before being used in the sewer, all poor material being rejected. Some of the points to be noted in making the inspection are as follows:

(1) The pipe should be straight, and true in shape.

(2) The pipe must have a hard-burned, strong internal structure showing incipient vitrification. Small pieces may be chipped out of occasional lengths to test this; and the color will also be a guide after the inspector has become thoroughly familiar with the make of pipe being used.

(3) The hub and socket ends of adjacent pipes should fit together well, leaving at least the spaces for cement given under Table I.

(4) There must not be on the lower half of the interior of the sewer any lumps, blisters, or excrescences. A few may be allowed

if not too large, if the pipe can be turned so as to bring them to the upper half.

(5) There must be no cracks extending into the body of the pipe, or of such nature as to weaken it materially. On tapping the pipe with a light hammer, if it does not give a clear ring, the presence of invisible cracks may be suspected.

(6) There must be no broken pieces of material size, from either the hub or the socket ends, nor any at all which cannot be turned to the upper half.

Nothing of human construction can be perfect, and sewer pipes are no exception to the rule. Hence the pipe inspector must have good judgment and considerable experience to draw the line properly between important and unimportant defects. In clause 25, Art. 93, of the sewer specifications given hereinafter, some definite rules are laid down to govern inspectors in this particular.

Vitrified pipe can be secured in 2, 2½, and 3-foot lengths. The longer the lengths, the fewer the joints, which is a material advantage.

33. Joints in Pipe Sewers. The joints are the weakest points in pipe sewers, and should be made with the utmost pains to secure as nearly as practicable an absolutely water-tight job. In Fig. 17, the upper joint shown illustrates the form commonly employed.

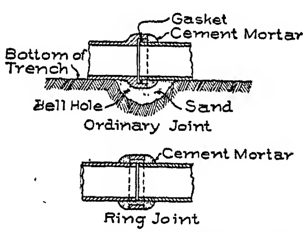


Fig. 17. Joints in Pipe Sewers.

In the bottom of the trench, which should be rounded to fit the under part of the sewer pipe, *bell-holes* are dug for all bells, to permit the joint on the under side of the pipe to be made properly, and to give the pipe a bearing on its full length instead of merely on the bells. Before the spigot end of the pipe to be laid is *entered* into the bell of the last pipe laid, it should be wrapped with a *gasket* of hemp, oakum, or jute, as shown in Fig. 17, so that the inverts of the two pipes will match in a smooth line when the pipe is entered, and so as to prevent the soft cement mortar from being forced up through the joint to project into the pipe. The gasket also assists in making the joint water-tight, especially if there is water in the trench. Disastrous results have often followed the omission of the gasket, which should always be used.

After the pipe is entered and brought exactly to grade, Portland cement mortar, mixed about 1 to 1 or 1 to 2 with sand, should be *calked* into the joint, *to fill it absolutely full,* and should be beveled off on the outside, as shown in the figure. Special care should be taken on the under side of the pipe. *Immediately* after placing the cement, the bell-hole should be packed *full* of sand, so as to support the cement on the under side of the pipe till it has set. It is best to keep the cementing back two or three lengths of pipe from the pipe laying, to avoid danger of the cement being broken in placing the next pipe.

Without the most careful watching of every joint during construction, the workmen are sure to slight the joints. An inspector should be kept constantly on the work.

In the lower part of Fig. 17 is shown the *ring joint,* formerly preferred by some engineers, but now very seldom used. It is more costly than the ordinary form.

Various joints have been invented and used to a limited extent, which include simple beveling of the ends of the pipe without using bells, the use of grooves at one end with corresponding projections at the other end, etc. Sometimes the exterior of the spigot end and the interior of the bells are grooved and made rough in the ordinary form of joint. This is an advantage in holding the cement, and in securing a water-tight job.

34. Cement Sewer-Pipe. Ever since the early use of pipe sewers in the latter half of the nineteenth century, cement pipe has been used to some extent for sewers; and recently there seems to be a revival and extension of its use. Experience has shown that cement is a very suitable material for making sewer pipe, and that cement pipes, when well made, of first-class materials, give excellent satisfaction for sewers, and are durable and not disintegrated by the sewage.

The manufacture of good cement sewer-pipe, however, cannot be successfully carried on by men who do not have the necessary skill, which is to be gained only by experience in this particular work; and even skilled manufacturers will not be successful unless both the cement and the sand used are of first-class quality, nor unless plenty of cement is used. Much poor cement pipe has been made, because these almost self-evident facts have not been understood; and in this way cement sewer-pipe has gained a bad reputation in many localities.

In general it may be said that the sand should be clean, sharp, and coarse, and that it should contain a considerable proportion of fine pebbles, smaller than a cherry-pit. Only the best Portland cement should be used, and the mortar should not be weaker than 1 to 3. The mixing must be very thorough, as also the tamping into the moulds.

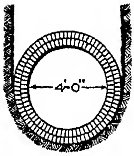


Fig. 18. Circular Brick Sewer, Ingersoll Run, Des Moines, Iowa.

Two general kinds of cement sewer-pipe are made. In one, just coming into use, the pipes are made continuously in the ditch. A form of moulds is used to give the correct shape and size, which can be forced ahead as the work progresses; and there are no joints. It is too soon yet to tell how successful this plan may be.

In the more common form of cement sewer-pipe, the pipes are made in a factory, in pieces of the same length as vitrified pipe. Usually, comparatively little water is used in mixing, in order to permit immediate removal of the pipe from the moulds. While such pipe are *curing* (setting), the omitted water must be supplied by frequently wetting them, or the process of setting and hardening cannot go on properly. Many cement sewer-pipes of this kind are spoiled in the curing.

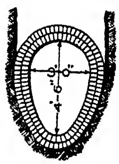


Fig. 19. Egg - Shaped Brick Combined Sewer.

Cement pipe are now made with bells for the joints, the same as vitrified pipe. The manufacture of specials, such as the Y-junctions required in such numbers for house connections, is still in unsatisfactory condition.

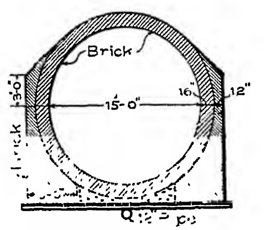


Fig. 20. Circular Brick Sewer with Subdrain, 64th Street Brooklyn, N. Y.

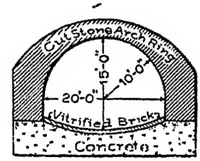


Fig. 21. Section of a Large Sewer in St. Louis, Mo.

The body of a cement sewer-pipe is of much weaker material than that of which vitrified pipe are made; and the thickness of cement pipe should be much greater than the thickness given in Table I for vitrified pipe.

35. Typical Cross-Sections of Large Sewers. In Figs. 18 to 25, inclusive, are shown some typical designs for sewers too large to be constructed of sewer pipe.

In Fig. 18, the common circular form is shown. This form is more economical to construct than any other when good foundations

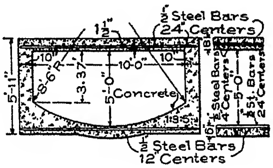


Fig. 22. Ingersoll Run Sewer with Low Headroom, Des Moines, Iowa.

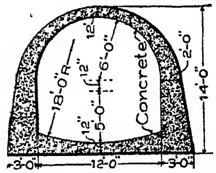


Fig. 23 Dry-Run Sewer, Waterloo, Iowa.

can be had, for the circle gives a larger area and velocity of flow when full than any other shape having the same circumference.

In the case of combined sewers, however, the dry-weather flow of sewage is so very small, in comparison with the size of the sewer, that it makes only a shallow, trickling stream of little velocity, and the sewer will not be self-cleansing. For such sewers, this difficulty can be overcome by the use of the egg-shape of sewer, shown in Fig. 19. This shape has a circular inv rt having a radius only half that of the top; and the depth and velocity of the dry-weather flow will be the same as in a circular sewer of this smaller radius, while at the same time the capacity in time of flood is equivalent to a much larger circle.

In Fig. 20, a favorite type of design for very large circular sewers

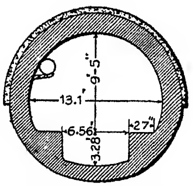


Fig. 24. Old Type of Main Sewers, Paris, France.

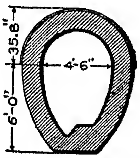


Fig. 25. New Type of Sewers, Paris, France.

is shown. For such large sewers, the upper half constitutes an arch, which exerts heavy pressures or thrusts horizontally outward against the sides of the sewer at the height of the center. To withstand these thrusts, the masses of masonry backing shown in the figure are added. This backing may be of brick, rubble-stone, or concrete masonry.

In the large sewers, too, it usually is not practicable to round the bottom of the trench to fit the circular shape, as is done for smaller sewers; and hence the flat foundation, also shown in the figure, is adopted. In soft materials, it often becomes necessary to drive piles to carry the weight of sewers.

In Fig. 21 is shown the favorite design for large sewers. For reasons given in discussing Fig. 20, the foundation is necessarily made flat; and with this shape of foundation, Fig. 21 will give a larger area and capacity for the same amount of material than Fig. 20, other conditions being the same. Also, Fig. 21 requires less head-room than Fig. 20 for the same capacity—which is often of great importance in the case of these large sewers. The invert of Fig. 21 is not so well suited to prevent deposits as that of Fig. 20; but in the case of these large sewers, there is usually a large flow even in dry weather, so that this point may be of little importance.

In Fig. 23 we have an example of the use of concrete for a large sewer of the general type shown in Fig. 21, and just discussed.

In Fig. 22 we have an extreme case of low headroom, secured by making the top an absolutely flat slab of concrete, reinforced with steel. In this case the bottom of the sewer was necessarily located at a very shallow depth below the street, while the required size of sewer was large.

Finally, in Figs. 24 and 25, are shown two typical cross-sections of the famous sewers of Pàris. The large main shown in Fig. 24 acts not only as a sewer, but also as a subway for the water mains and for other purposes. The entire ordinary flow of sewage is confined within the *cunette*, or comparatively small channel shown in the bottom. The ledge on each side serves for the passage of workmen and of cleaning carts, flushing devices, etc. The section shown in Fig. 25 is a later type, and is more nearly self-cleansing. The dirt in the streets is washed into these sewers by the use of hose, and special conveniences for cleaning it out of the sewers are needed.

36. Junction-Chambers for Large Sewers. Where two or more large sewers join, special difficulties present themselves, in providing supports for the partial arches whose supports are cut away in making the junction. It is usually necessary, when the sewers are large, to build a masonry chamber enclosing the entire junction, and with a self-supporting roof spanning all the sewers.

Various designs for such junction-chambers are used, but the most common type is illustrated in Fig. 26. Here a *bell-mouth arch* is used to span the opening, the case being the junction of three of the Chicago intercepting sewers (see Fig. 5). Sometimes *flat roofs* are used, supported by steel beams or made of reinforced concrete.

The bottoms of such junctions are the *mathematical intersections*, executed in masonry, of the lower halves of the sewer channels; and for sewers not too large, the upper halves may sometimes be built in a similar way, or with *vault ribs*, as in the roofs of old cathedrals.

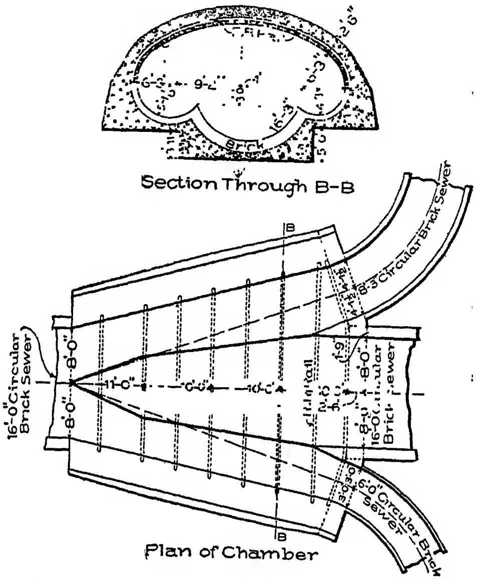


Fig. 26. Junction of Brick Sewers, Lawrence and Sheridan Avenues, Chicago, Ill.

37. Brick Sewers. It has already been stated that brick is the favorite material for sewers too large to be made of pipe, the dividing line usually being drawn at 30 inches to 36 inches diameter. Brick present many advantages for sewer work, including their moderate cost, their durability, and their small size and regular shape, which enable them to be readily handled and used in building sewers of any desired cross-section, with comparatively smooth and true interior surfaces.

Sewer brick, as those suitable for sewer construction are commonly called, should be harder burned than ordinary building brick, to enable them to stand the wear from the flow of sewage, and to insure against disintegration. They need not, however, be as hard burned as No. 1 paving brick, and hence constitute an intermediate grade between building brick and pavers. Sewer brick should be uniform in size, and of regular, true shape, so as to permit of being laid with thin joints, to form smooth, true surfaces. They should be carefully inspected on the work just before being used, and all defective brick

thrown out. The common size for sewer brick approximates $8\frac{1}{2}$ by 4 by $2\frac{1}{4}$ inches.

In the sewer, the brick are laid in rings, as shown in Figs. 18 and 19, with the 4-inch dimension radial and the $8\frac{1}{2}$-inch dimension lengthwise of the sewer. Care should be taken to *break joints* in each ring. The brick should be laid in Portland cement mortar, made of at least 1 part of cement to 3 parts of clean, sharp sand of medium-sized grains. Pebbles should be screened out of the sand so as to permit thin joints. All joints should be filled *full* of mortar, the brick being laid with *shove joints,* to make a practically water-tight job. The outside ring of the invert should be laid against a layer of 1 to 2 Portland cement mortar; and the outside of the arch (or upper half of the sewer) should be plastered with the same mortar, to keep out ground water. Similarly, to prevent leakage of sewage, the entire interior surface of the sewer should be plastered with the same mortar, or else thoroughly washed with at least two coats of liquid cement, after the joints have been carefully pointed and smoothed. Even with the utmost care, it will be found impossible to secure absolute watertightness; and the difficulties will be especially great when ground water and soft materials are encountered in the trench.

Up to 6 or 7 feet diameter, two rings of brick are usually sufficient. In fact, for the smaller sizes of brick sewers, one ring would be amply strong with firm foundations; but it is difficult to make the sewer sufficiently tight when only one ring is used, because all joints extend entirely through. Sometimes an exterior layer of concrete may be used to meet this objection, at least for the lower half of the sewer; or an outside ring of brick may be used for the invert only. Sewers larger than 6 or 7 feet in diameter usually require three rings of brick; and more are needed for very large sewers, for which the number required must be calculated for each particular case to suit the special conditions.

38. Concrete Sewers. Of late years, concrete has frequently been employed in preference to other kinds of masonry for many purposes, of which sewer construction is one. Its advantages for sewers are many. The following may be mentioned:

First, and foremost, the cost is usually less than the cost of brick masonry.

Second, the concrete exactly fits the irregularities of the excavation, giving better foundations.

Third, sewers built of concrete constitute a solid structure without joints, and hence are less liable to uneven settlement.

Fourth, there are no joints, as in brickwork, to be made watertight, though, on the other hand, it is not easy to make the body of the concrete entirely impervious to seepage.

Fifth, the concrete can be readily moulded to any desired shape of sewer.

Sixth, the concrete can be made by comparatively unskilled workmen, if skilled foremen are employed.

Concrete may be used for foundations, as shown in Figs. 20 and 21; for the backing of brick sewer rings; and in various other combinations with brick; or it may be used for the entire sewer, as in Figs. 22 and 23.

Reinforced concrete, or concrete reinforced with steel rods, to prevent cracks from tension stresses, has opened up of late years entirely new possibilities in sewer construction, of which Fig. 22 is an example.

It has been reported that the concrete invert of the large St. Louis sewer shown in Fig. 21 has shown surface pitting and disintegration from the effects of the sewage. This is a trouble which does not appear to have been experienced elsewhere, and hence is presumably uncommon, and would seem due most probably to poor materials or poor workmanship. Danger from this source could be prevented by lining the concrete sewer with one ring of vitrified paving brick.

FORMULÆ AND DIAGRAMS FOR COMPUTING FLOW IN SEWERS

39. Formulæ for Computing Flow in Sewers. It has already been stated that more than 99.8 per cent of even sanitary sewage is simply ordinary water which has been added to the foul wastes to assist in removing them. Hence the mathematical formulæ for the flow of sewage are the same as those for the flow of water. They may be studied in detail in the instruction paper on Hydraulics.

Two general hydraulic formulæ have commonly been employed in sewer computations, as follows:

(1) *Weisbach's Formula.* The older computations were generally based on Weisbach's formula, which is as follows:

$$v = \frac{\sqrt{2gh}}{\sqrt{1 + e + c\frac{l}{d}}}.$$

In the above formula,

v = Average velocity of flow, in feet per second.
g = Acceleration due to gravity = 32.2 ft. per second.
h = Fall of sewer, in feet.
e = Coefficient of entrance = 0.505.
c = Coefficient of friction in pipe $= 0.0144 + \frac{0.0169}{\sqrt{v}}$.
l = Length of pipe, in feet.
d = Diameter of pipe, in feet.

Weisbach's formula has been much used for sewer computations, for the reason that Mr. Baldwin Latham, in the first treatise on Sanitary Engineering worthy the name (1873), published extensive tables of flow, calculated from this formula, which made sewer computations very simple. Hence it was easier for later engineers simply to make use of these tables than to compute new ones of their own.

(2) *Kutter's Formula.* In later hydraulic computations, it has generally been considered that Kutter's formula gives the most reliable results. It is as follows:

$$v = c\sqrt{RS} = \left\{ \frac{41.66 + \frac{1.811}{n} + \frac{.00281}{s}}{1 + \left(41.66 + \frac{.00281}{s}\right)\frac{n}{\sqrt{R}}} \right\} \sqrt{RS}.$$

In this formula,

v = Average velocity of flow, in feet per second.

R = Mean hydraulic radius in feet = Area of cross-section of stream in square feet, *divided by* wetted perimeter, in feet, of length of portion of circumference of channel wet by the stream. (NOTE.—For circular pipe sewers, $R = \frac{1}{4}$ of the diameter when the pipe is flowing either full or half-full.)

S = Slope of the sewer $= \frac{\text{Fall}}{\text{Length}}$.

n = Coefficient of roughness, varying with the roughness of the channel.

For pipe sewers it is common to assume that $n = 0.013$; and for brick sewers, that $n = 0.015$. For cement pipe sewers, the roughness might be considered intermediate between these values of n; but $n = 0.013$ is generally used for them as well as for clay pipe. New and perfectly clean channels

would not be so rough as indicated by these numbers; but the growths a d deposits which may accumulate in sewers render it wise to adopt the above values for n.

Both the above sewer formulæ give merely the average velocities (v) of flow. *To obtain the discharge in cubic feet per second, we must multiply "v" by the area in square feet of the cross-section of the stream of sewage.*

Kutter's formula gives less capacities for pipe sewers than Weisbach's for the small sizes, up to about 18 inches' diameter. It will be on the safe side to adopt Kutter's formula; and this is now very generally done, though actual gaugings of small-pipe sewers either new or in very good condition, may often show greater velocities and capacities than the formula would indicate, when the values of n above given are adopted.

In this paper, Kutter's formula will be adopted as the basis of all calculations of the flow of sewers.

40. Diagram of Discharges and Velocities of Circular Pipe Sewers Flowing Full. Direct numerical computations of flow in sewers from the formulæ given above, would be very laborious and tedious. The work may be very greatly simplified by the use of tables or diagrams. Diagrams are more convenient than tables, and are adopted for this paper. With their aid, computations of flow in sewers are very easy and short.

Fig. 27 is such a diagram, giving the capacities and velocities of circular *vitrified* pipe sewers flowing full. *Cement* pipe sewers would probably have discharges and velocities somewhat less than those shown in this figure.

TO USE THE DIAGRAM

(A) *When the diameter of the pipe and the grade are given, to find the discharge and the velocity.*

(1) Look along the bottom horizontal line till the grade is found, interpolating by the eye, if necessary, between the grades marked on the diagram. (2) Find the point where the vertical line through the given grade intersects the inclined line marked with the given diameter of sewer. (3) Trace horizontally through this point, interpolating by the eye, if necessary, between the horizontal lines on the diagram; and read the discharge of the pipe running full, on the left side of the diagram in cubic feet per second, or on the right side of the diagram in gallons per 24 hours. (4) If the velocity is desired, it can be determined by noting where the point (found in 2, above) of intersection of the given grade and diameter lines falls with reference to the inclined lines marked with the different velocities, estimating by the eye the decimals of a foot per second.

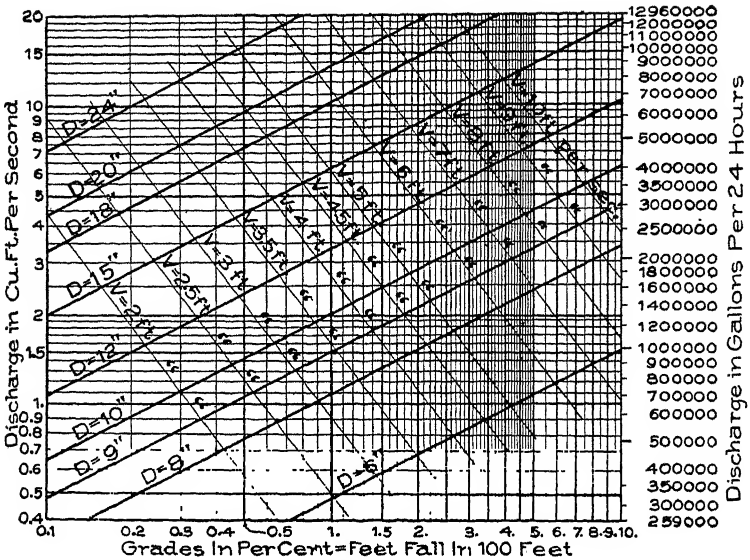


Fig. 27. Discharges and Velocities of Circular Vitrified Pipe Sewers Flowing Full. By Kutter's Formula (n=0 013).

(*B*) *When the grade and the required discharge are given, to find the necessary diameter of pipe, and the velocity.*

(1) Look along the bottom horizontal line till the given grade is found, interpolating by the eye, if necessary, between the grades marked on the diagram. (2) Find the intersection of the vertical line through this grade with the horizontal line through the given discharge, finding the discharge on the left of the diagram if it is given in cubic feet per second, or on the right if it is given in gallons per 24 hours. (3) Note between which two diameter lines this point of intersection falls, and take the diameter line nearest as that required. (4) Also note the position of the point of intersection with reference to the velocity lines, and so estimate the velocity, interpolating by the eye between the inclined velocity lines.

(*C*) *When the velocity and diameter are given, to find the grade and discharge.*

(1) Find the intersection of the given diameter line with the given velocity line, interpolating by the eye, if necessary. (2) Then vertically downward to the bottom of the diagram from this point of intersection, read the required grade; and horizontally to the left side or to the right side of the diagram, read the discharge, interpolating by the eye in each case, if necessary.

All other cases may be solved by similar obvious methods.

EXAMPLES

Example 1.—What will be the discharge and velocity of a 15-inch pipe sewer laid to a 0.2 per cent grade?

Solution. See *A*, above. From the intersection of the vertical 0.2 per cent grade line with the inclined 15-inch diameter line, we read horizontally to the left the discharge of 2.8 cu. ft. per second, or to the right, of 1,850,000 gallons per 24 hours. We further note that the point of intersection of the 0.2 per cent grade line with the 15-inch diameter line falls between the 2.0 and the 2.5 ft. per second velocity lines, and by the eye we estimate the velocity to be 2.3 ft. per second.

Example 2. See *B*, above. What size of pipe sewer laid at a grade of 0.5 per cent will be required to carry an average flow of 200,000 gallons of sewage per day, the maximum rate of discharge being three times the average? (NOTE.—Hence use 600,000 gallons discharge in solving the example.) Also, what will be the velocity?

Answer. Required diameter of sewer, 9 inches; velocity of flow, about 2.3 ft. per second.

Example 3. See *C*, above. If the minimum allowable velocity of flow is 2 ft. per second when a sewer flows full, what minimum grade will be required to produce this velocity in a 12-inch sewer?

Answer. 0.23 per cent minimum grade.

Example 4. If an outlet sewer serves 20,000 people, each person contributes 100 gallons per day, and the maximum rate of flow is 3 times the average, what size of sewer will be required, if its grade is 0.25 per cent?

Answer. 24 inches diameter.

Example 5. If an 8-inch pipe sewer is laid at a 0.45 per cent grade, what will be the discharge and the velocity when it flows full?

Answer. 480,000 gallons per day; 2.1 ft. per second.

Example 6. A storm pipe sewer drains 10 acres, and should be able to carry 1.5 cu. ft. per second per acre. Its grade is 0.5 per cent. What diameter will be required?

Answer. 24 inches diameter.

41. Diagram of Discharges and Velocities of Circular Brick and Concrete Sewers Flowing Full. Fig. 28 is the diagram for circular brick and concrete sewers, corresponding to Fig. 27 for pipe sewers, and is used in the same way.

TO USE THE DIAGRAM

(*A*) *When the diameter of the pipe and the grade are given, to find the discharge and the velocity:*

(1) Look along the bottom horizontal line till the grade is found, interpolating by the eye, if necessary, between the grades marked on the diagram. (2) Find the point where the vertical line through the given grade intersects the inclined line marked with the given diameter of sewer. (3) Trace hori-

zontally through this point, interpolating by the eye, if necessary, between the horizontal lines on the diagram; and read the discharge of the pipe running full, on the left side of the diagram in cubic feet per second, or on the right side of the diagram in gallons per 24 hours. (4) If the velocity is desired,

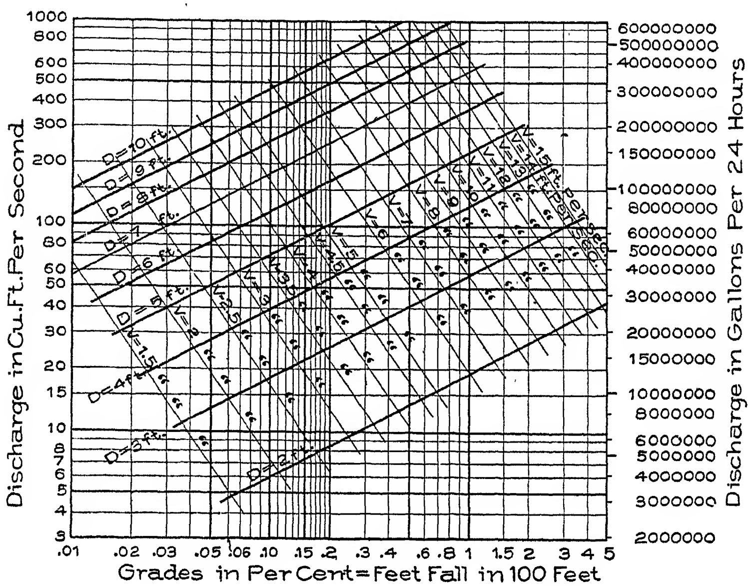


Fig. 28. Discharges and Velocities of Circular Brick and Concrete Sewers Flowing Full By Kutter's Formula (n=0.015).

it can be determined by noting where the point (found in 2, above) of intersection of the given grade and diameter lines falls with reference to the inclined lines marked with the different velocities, estimating by the eye the decimals of a foot per second.

(*B*) *When the grade and the required discharge are given, to find the necessary diameter of pipe, and the velocity.*

(1) Look along the bottom horizontal line till the given grade is found, interpolating by the eye, if necessary, between the grades marked on the diagram. (2) Find the intersection of the vertical line through this grade with the horizontal line through the given discharge, finding the discharge on the left of the diagram if it is given in cubic feet per second, or on the right if it is given in gallons per 24 hours. (3) Note between which two diameter lines this point of intersection falls, and take the diameter line nearest as that required. (4) Also note the position of the point of intersection with reference to the velocity lines, and so estimate the velocity, interpolating by the eye between the inclined velocity lines.

(*C*) *When the velocity and diameter are given, to find the grade and discharge.*

(1) Find the intersection of the given diameter line with the given velocity line, interpolating by the eye, if necessary. (2) Then vertically

downward to the bottom of the diagram from this point of intersection, read the required grade; and horizontally to the left side or to the right side of the diagram, read the discharge, interpolating by the eye in each case, if necessary.

All other cases may be solved by similar obvious methods.

EXAMPLES

Example 7. What size of circular brick or concrete sewer laid to a 0.2 per cent grade will be required to carry a storm sewage flow of ¾ cu. ft. per second per acre from one square mile of drainage area, and what will be the velocity?

Solution. See *B*, above. 1 square mile = 640 acres. The capacity required is 640 × ¾ = 480 cu. ft. per second, which we find on the left of Fig. 28 just below the 500 cu. ft. per second horizontal line, interpolating by eye. We next find the 0.2 per cent grade line at the bottom of the diagram, and locate the point of intersection of this vertical 0.2 per cent grade line with the horizontal 480 cu. ft. per second line already found above. This point of intersection comes nearly on the 9 feet inclined diameter line, and between the seven and eight feet per second inclined velocity lines.

Answer. Diameter of sewer required, 9 feet. Velocity = 7.6 ft. per second.

Example 8. What will be the minimum grade for a 60-inch brick or concrete sewer, if the minimum velocity allowed when flowing full is 3 ft. per second?

Answer. See *C*, above. 0.067 per cent grade.

Example 9. How large a population, contributing 75 gallons per capita per day of sanitary sewage, on the average (the maximum flow being 3 times the average), can be served by a 48-inch circular brick sewer, laid to a 0.06 per cent grade; and what will be the velocity of flow? (NOTE: Find the capacity as in *A*, above; and then divide by 3 times the average per capita amount per day.)

Answer. 89,000 population. 2.4 ft. per second.

Example 10. What will be the grade required to force a flow of 500 cu. ft. per second through a 96-inch circular brick sewer?

Answer. 0.38 per cent grade.

42. Diagram of Discharges and Velocities of Egg-Shaped Brick and Concrete Sewers Flowing Full. Fig. 29 is the diagram for egg-shaped brick sewers, corresponding to Fig. 27 for circular pipe sewers, and to Fig. 28 for circular brick and concrete sewers.

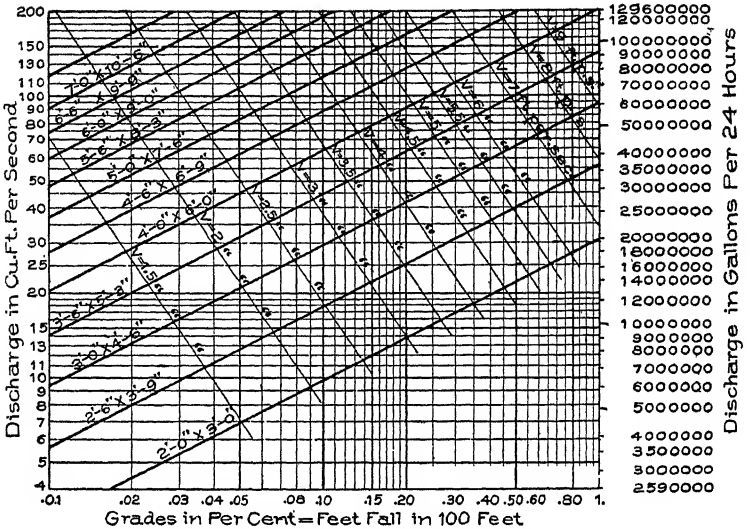


Fig. 29. Discharges and Velocities of Egg-Shaped Brick and Concrete Sewers Flowing Full. By Kutter's Formula (n=0.015).

TO USE THE DIAGRAM

(*A*) *When the diameter of the pipe and the grade are given, to find the discharge and the velocity.*

(1) Look along the bottom horizontal line till the grade is found, interpolating by the eye, if necessary, between the grades marked on the diagram. (2) Find the point where the vertical line through the given grade intersects the inclined line marked with the given diameter of sewer. (3) Trace horizontally through this point, interpolating by the eye, if necessary, between the horizontal lines on the diagram; and read the discharge of the pipe running full, on the left side of the diagram in cubic feet per second, or on the right side of the diagram in gallons per 24 hours. (4) If the velocity is desired, it can be determined by noting where the point (found in 2, above) of intersection of the given grade and diameter lines falls with reference to the inclined lines marked with the different velocities, estimating by the eye the decimals of a foot per second.

(*B*) *When the grade and the required discharge are given, to find the necessary diameter of pipe, and the velocity.*

(1) Look along the bottom horizontal line till the given grade is found, interpolating by the eye, if necessary, between the grades marked on the diagram. (2) Find the intersection of the vertical line through this grade with the horizontal line through the given discharge, finding the discharge on the left of the diagram if it is given in cubic feet per second, or on the right if it is given in gallons per 24 hours. (3) Note between which two diameter lines this point of intersection falls, and take the diameter line nearest as that required. (4) Also note the position of the point of intersection with reference to the velocity lines, and so estimate the velocity, interpolating by the eye between the inclined velocity lines.

(*C*) *When the velocity and diameter are given, to find the grade and discharge.*

(1) Find the intersection of the given diameter line with the given velocity line, interpolating by the eye, if necessary. (2) Then vertically downward to the bottom of the diagram from this point of intersection, read the required grade; and horizontally to the left side or to the right side of the diagram, read the discharge, interpolating by the eye in each case, if necessary.

All other cases may be solved by similar obvious methods.

EXAMPLES

Example 11. What will be the discharge and velocity of flow of a 4 by 6-feet egg-shaped brick or concrete sewer flowing full and laid to a 0.4 per cent grade?

Solution. See A, above. Find the 0.4 per cent grade line at the bottom of Fig. 29, and locate the point of intersection of the vertical line through this point with the inclined 4 by 6 dimension line. Then tracing horizontally to the left, we estimate by the eye 128 cu. ft. per second for the discharge. We also note that the point of intersection of the vertical 0.4 per cent grade line with the inclined 4 by 6 dimension line found above, is practically on the inclined 7 ft. per second velocity line.

Answer. Discharge, 128 cu. ft. per second. Velocity, 7 ft. per second.

Example 12. What will be the size of egg-shaped brick or concrete sewer required to carry a storm flow of ½ cu. ft. per second per acre from a drainage area of ½ square mile (= 320 acres), the grade being 0.3 per cent?

Answer. See *B*, above. 4 ft. 6 in. by 6 ft. 9 in.

Example 13. A 6-foot circular sewer and a 5 by 7 ft. 6-in. egg-shaped sewer have nearly the same area of cross-section. If both are laid to a 0.2 per cent grade, find the discharge and velocity of each when flowing full. (NOTE: Solve by Figs. 28 and 29. See *A*, above.)

Answer. Discharge, 165 cu. ft. per second; and velocity, 5.8 ft. per second, for the circular sewer; and discharge 163 cu. ft. per second; and velocity, 5.7 ft. per second, for the egg-shaped sewer.

NOTE: Although the egg-shaped sewer has a slightly smaller velocity when both are flowing full, it has a materially greater velocity than the circular sewer for small depths of flow.

Example 14. If the minimum allowable velocity of flow in storm sewers is 3 ft. per second, find the minimum allowable grades for 2 ft. by 3 ft., 4 ft. by 6 ft., and 6 ft. by 9 ft. egg-shaped sewers, respectively.

Answer. See *C*, above. 0.20, 0.08, and 0.05 per cent, respectively.

43. Diagram of Discharges and Velocities in Circular Sewers at Different Depths of Flow. The diagrams so far given show the discharges and velocities in sewers *flowing full.* It often, however, is necessary to be able to calculate the discharge and the velocity when the sewer flows only *partially* full.

For *circular sewers,* the discharges and velocities, when flowing only *partially full,* can readily be determined by the use of the diagram, Fig. 30, in connection with Figs. 27 and 28.

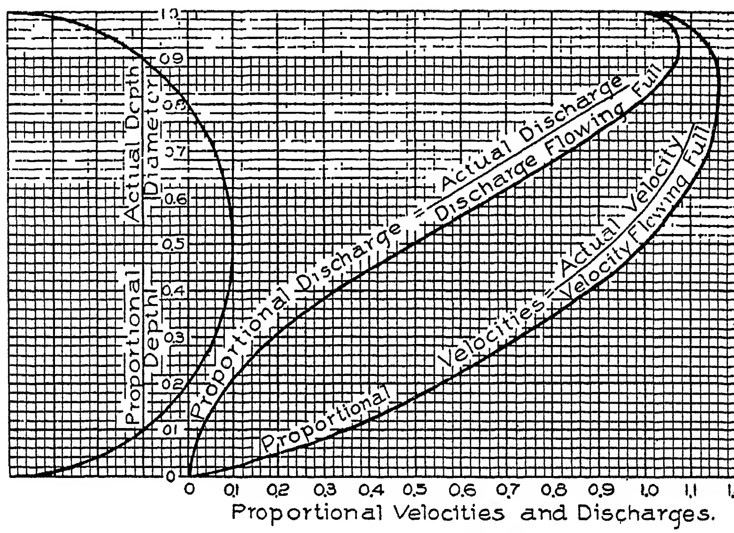


Fig. 30. Diagram Showing Changes in Velocity and Discharge in Circular Sewers for Different Depths of Flow.

TO USE THE DIAGRAM

(*A*) *When the depth of flow is given, together with the diameter and grade of the sewer, to determine the discharge and the velocity.*

(1) By Fig. 27 if a pipe sewer, or by Fig. 28 if a brick or concrete sewer, determine the *discharge* and *velocity* of the sewer *flowing full.* (2) Divide the given depth of flow by the given diameter, to determine the *proportional depth* of flow; and find this proportional depth on the vertical scale towards the left of Fig. 30, interpolating by the eye, if necessary. (3) Find the intersection of the horizontal line through the proportional depth (found in 2, above), first, with the *proportional discharge line,* and, second, with the *proportional velocity line,* in Fig. 30; and read off at the bottom of the diagram vertically below these intersection points, the *proportional discharge* and the *proportional velocity.* (4) Multiply the *discharge and velocity flowing full* (found in 1, above), by the *proportional discharge* and *proportional velocity*

found in 3, above), and the products will be the required *actual discharge and actual velocity, for the given depth of flow.*

(*B*) *When the actual discharge is given, together with the diameter and grade of the sewer, to find the depth and velocity of flow.*

(1) By Fig. 27 if a pipe sewer, or by Fig. 28 if a brick or concrete sewer, determine the *discharge of the sewer flowing full.* (2) Divide the given discharge by the discharge flowing full, to determine the *proportional discharge;* and find this along the bottom of the diagram in Fig. 30, interpolating by the eye, if necessary. (3) Find the intersection of the vertical line through the proportional discharge (found in 2, above) with the *proportional discharge curve* in Fig. 30; and horizontally to the left, read off on the vertical scale near the left of the diagram the *proportional depth* of flow. (4) Multiply the diameter of the sewer by the proportional depth, and the product will be the *actual depth of flow for the given discharge.* (5) The actual velocity can now be found as described above for case *A*.

All other cases than *A* and *B* can be readily solved by similar obvious methods.

EXAMPLES

Example 15. What will be the actual discharge and velocity of flow in a 48-inch circular brick sewer laid to a 0.15 per cent. grade, and flowing 6 inches deep?

Solution. See *A*, above. (1) By Fig. 28, with the sewer flowing full, the discharge would be 30,000,000 gallons per day, and the velocity 3.8 ft. per second. (2) $\frac{6 \text{ inches}}{48 \text{ inches}} = 0.12+ =$ proportional depth of flow, which we find on the vertical scale near the left of Fig. 30. (3) Horizontally opposite the point found in 2, we locate points on the proportional discharge curve and the proportional velocity curve in Fig. 30; and vertically beneath these points we read at the bottom of the diagram, 0.04 = proportional discharge, and 0.40 = proportional velocity. (4) $0.04 \times 30{,}000{,}000$ gallons = 1,200,000 gallons per day = actual discharge for 6 inches depth of flow; and $0.40 \times 3.8 = 1.5$ ft. per second = actual velocity for 6 inches depth of flow.

Example 16. An 8-inch pipe sewer, laid to a 0.40 per cent grade, is to carry the sewage of 500 people contributing 100 gallons each per day. What will be the average depth and velocity of flow?

Solution. See *B*, above. (1) By Fig. 27, the discharge and velocity flowing full would be respectively 450,000 gals. per day, and 1.9 ft. per second. (2) The actual discharge is $500 \times 100 = 50{,}000$ gals. per day, and hence the *proportional discharge* is $\frac{50{,}000}{450{,}000} = 0.11$. We find this proportional discharge along the bottom line of Fig. 30, interpolating by eye. (3) Vertically above the 0.11 proportional velocity,

we find a point on the proportional discharge curve; and tracing horizontally to the left, we there read off the proportional depth = 0.225. (4) 0.225 × 8 = 1.8 inches = the actual depth of flow for the given discharge. (5) Horizontally to the right from the 0.225 proportional depth, we find a point on the proportional velocity line; and vertically beneath this point we read off at the bottom of the diagram, proportional velocity = 0.60. Then 0.60 × 1.9 (see 1, above) = 1.1 ft. per second = actual velocity for the given depth.

Example 17. What will be the discharge and velocity of a 12-inch pipe sewer laid to a 0.25 per cent grade when flowing 4 inches deep?

See *A*, above.

Answer. Discharge, 250,000 gals. per day; velocity, 1.7 ft. per second.

Example 18. What will be the depth and velocity of flow in a

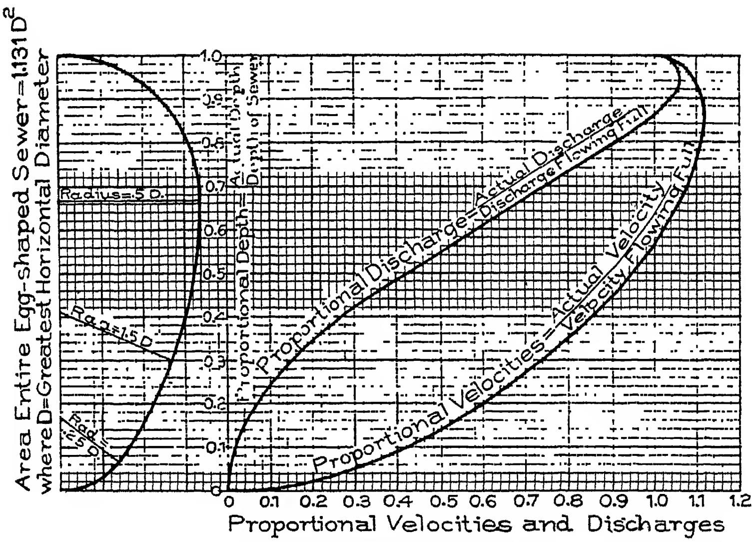


Fig. 31. Diagram Showing Changes in Velocity and Discharge in Egg-Shaped Sewers for Different Depths of Flow.

15-inch pipe sewer, laid at a 0.2 per cent grade, carrying 1,000,000 gallons of sewage per day?

See *B*, above.

Answer. Depth, 8 inches; velocity, 2.3 ft. per second.

44. Diagram of Discharges and Velocities in Egg-Shaped Sewers at Different Depths of Flow. For egg-shaped sewers, the discharges and velocities, when flowing *partially full,* can readily be determined by the diagram, Fig. 31, used in connection with Fig. 29.

TO USE THE DIAGRAM

(*A*) *When the depth of flow is given, together with the diameter and grade of the sewer, to determine the discharge and the velocity.*

(1) By Fig. 29, determine the *discharge and velocity* of the sewer *flowing full.* (2) Divide the given depth of flow by the given height to determine the *proportional depth* of flow, and find this proportional depth on the vertical scale towards the left of Fig. 31, interpolating by the eye, if necessary. (3) Find the intersection of the horizontal line through the proportional depth (found in 2, above), first, with the *proportional discharge line,* and, second, with the *proportional velocity* line, in Fig. 31; and read off at the bottom of the diagram, vertically below these intersection points, the *proportional discharge,* and the *proportional velocity.* (4) Multiply the *discharge and velocity flowing full* (found in 1, above), by the *proportional discharge and proportional velocity* (found in 3, above), and the products will be the required *actual discharge and actual velocity for the given depth of flow.*

(*B*) *When the actual discharge is given, together with the diameter and grade of the sewer, to find the depth and velocity of flow.*

(1) By Fig. 29, determine the *discharge of the sewer flowing full.* (2) Divide the given discharge by the discharge flowing full, to determine the *proportional discharge,* and find this along the bottom of the diagram in Fig. 31, interpolating by the eye, if necessary. (3) Find the intersection of the vertical line through the proportional discharge (found in 2, above), with the *proportional discharge curve* in Fig. 31, and horizontally to the left, read off on the vertical scale near the left of the diagram the *proportional depth* of flow. (4) Multiply the height of the sewer by the proportional depth, and the product will be the *actual depth of flow for the given discharge.* (5) The actual velocity can now be found as described above for case *A*.

All other cases than *A* and *B* can be readily solved by similar obvious methods.

EXAMPLES

Example 19. What will be the discharge and velocity in an egg-shaped brick or concrete sewer 3 ft. by 4 ft. 6 in., laid to a 0.15 per cent grade, and flowing 12 inches deep?

See *A*, above.

Solution. (1) By Fig. 29, discharge and velocity flowing full = 36 cu. ft. per second, and 3.45 ft. per second, respectively. (2) The proportional depth $= \frac{12}{54} = 0.22$, which we find at left of Fig. 31. (3) We locate the intersections of the horizontal line through the 0.22 proportional depth with the proportional discharge and proportional velocity curves, respectively; and vertically below these points we read off, at the bottom of the diagram, proportional discharge = 0.08, and proportional velocity = 0.63. (4) $36 \times 0.08 = 2.9$ cu. ft. per second = actual discharge; $3.45 \times 0.63 = 2.2$ ft. per second = actual velocity.

Answer. Discharge = 2.9 cu. ft. per second; velocity = 2.2 ft. per second.

Example 20. What will be the depth and velocity of flow in an egg-shaped brick or concrete sewer 5 ft. by 7 ft. 6 in. dimensions, laid to a 0.10 per cent grade, and carrying 30 cu. ft. per second flow of sewage?

See *B*, above.

Solution. (1) By Fig. 29, the discharge and velocity flowing full = 117 cu. ft. per second and 4.05 ft. per second, respectively. (2) Proportional discharge = $\frac{30}{117}$ = 0.26–, which find at bottom of Fig. 31. (3) Vertically above the 0.26 proportional discharge, we locate a point on the proportional discharge curve in Fig. 31, and horizontally to the left from this point read off the proportional depth = 0.39. (4) $90 \times 0.39 = 35$ inches = actual depth of flow. (5) Horizontally to the right along the 0.39 proportional depth line, we locate a point on the proportional velocity line; and vertically beneath this, we read off, at the bottom of the diagram, proportional velocity = 0.845. Then $4.05 \times 0.845 = 3.4$ ft. per second = actual velocity.

Answer. Depth of flow = 35 inches; velocity = 3.4 ft. per second.

Example 21. What will be the discharge and velocity in an egg-shaped brick or concrete sewer 2 ft. by 3 ft. dimensions, laid to a 0.50 per cent grade, flowing 18 inches deep?

See *A*, above.

Answer. Discharge = 5,900,000 gals. per day; velocity = 4.5 ft. per second.

Example 22. What will be the depth and velocity of flow in an egg-shaped brick or concrete sewer 3 ft. 6 in. by 5 ft. 3 in. dimensions, laid to a 0.08 per cent grade, carrying 25 cu. ft. per second of sewage?

See *B*, above.

Answer. Depth of flow = 39 inches; velocity of flow = 2.9 ft. per second.

GENERAL EXAMPLES FOR PRACTICE WITH FIGS. 27-31

45. The solution of the following general examples will further familiarize the student with the principles thus far explained.

Example 23. A 24-inch sewer is to be laid to a 0.25 per cent grade, and may be made of vitrified sewer pipe or of brick. Compare the discharges and velocities obtained with the two materials. (NOTE: Use Figs. 27 and 28.)

Answer. With sewer pipe, discharge = 7,200,000 gals. per day; velocity = 3.6 ft. per second.
With brick, discharge = 6,000,000 gals. per day; velocity = 3 ft. per second.

Example 24. A combined sewer, laid to a 0.15 per cent grade, drains an area requiring either a 3-foot circular or a 2 ft. 6 in. by 3 ft. 9 in. egg-shaped brick sewer. (These sizes have the same cross-sectional area, and nearly the same discharges and velocities, when flowing full.) The dry-weather flow of sewage will be only 1,000,000 gallons per day. Calculate the dry-weather depth and velocity of flow with each design. (NOTE: Use Figs. 28 and 30, and Figs. 29 and 31.)

Answer. With circular sewer, depth = 6.1 inches; velocity = 1.6 ft. per second.
With egg-shaped sewer, depth = 9.2 inches; velocity = 1.9 ft. per second.

Example 25. In a 10-inch pipe sewer, laid to a one per cent grade, the maximum depth of flow observed was 7 inches; and the minimum, 2 inches. What were the corresponding discharges? (NOTE: Use Figs. 27 and 30.)

Answer. Maximum discharge = 1,100,000 gals. per day;
Minimum " = 120,000 " " "

Example 26. What size of circular sewer laid to a 0.08 per cent grade will be required to carry the sanitary sewage of a city of 100,000 population, with an average flow of sewage of 150 gallons per capita per day, the maximum rate of flow being three times the average?

Answer. 5 ft. 3 in. diameter.

Example 27. What size of egg-shaped combined sewer, laid to a 0.07 per cent grade will be required to carry a storm sewage flow of 0.5 cu. ft. per second per acre from a drainage area of 320 acres?

Answer. 6 ft. by 9 ft.

46. Summary of Laws of Flow in Sewers. The principles discussed in Articles 38 to 44, inclusive, may be briefly summarized as follows:

(1) The laws of flow for sewage are the same as for water.

(2) Kutter's formula is generally considered most reliable for calculating the flow in sewers, though complicated to use directly.

(3) In Kutter's formula, the values of the coefficient of roughness generally used for sewer computations, are $n = 0.013$ for pipe sewers, and $n = 0.015$ for brick and concrete sewers.

(4) Sewer diagrams greatly simplify sewer computations, and are presented in Figs. 27 to 31, inclusive, for circular and egg-shaped sewers, with full instructions for use.

(5) In Fig. 30, the laws of flow for different depths of flow in

circular sewers are shown. An examination of the diagram brings out this important law:

In circular sewers flowing half-full, the velocity is the same as when the sewer flows full; and hence the discharge flowing half-full is just half the discharge flowing full.

(6) Figs. 30 and 31 also show the following important law of flow:

In a sewer of any shape, not flowing under pressure, the maximum discharge and velocity will occur, not with the sewer flowing full, but with it flowing a little less than full.

This is due to the increased friction against the top of the sewer when it flows full. Owing to this law, no sewer can flow full without being under pressure.

(7) In the case of combined sewers having a dry-weather flow very small as compared with the storm flow, egg-shaped sewers give materially greater depths and velocities of dry-weather flow than circular sewers.

CALCULATIONS OF SIZES AND MINIMUM GRADES OF SEPARATE SANITARY SEWERS

47. Minimum Sizes of Sanitary Sewers. In the early construction of sewers, previous to the last half of the 19th century, the laterals and sub-mains were usually made very much larger than the amount of sewage would require, with the idea, apparently, that the bigger the sewer the better. Such badly proportioned sewers were in great danger of stoppages from the inability of the shallow, trickling stream to carry along the solid matter. In fact, the sewers were expected to form deposits, and were purposely made large to hold a large amount of deposit and to enable men to enter for the purpose of cleaning them. Disastrous sanitary experience with such foul sewers made it apparent that there was just as much danger from making the sewers too large as from making them too small, especially in the case of sanitary sewers. Such sewers should be made small enough to give a good depth and velocity of flow.

Sanitary sewers should not be made small enough, however, to cause frequent stoppages by catching articles which have been admitted into them through the house connections. House owners are often reprehensibly negligent in putting into their plumbing fixtures,

articles which should be carefully excluded. On this account, the size of house connections should be restricted to 4 inches.

An 8-inch sewer pipe will practically always carry freely, even crosswise, any article which can come lengthwise around the traps and bends in 4-inch soil-pipes and house connections Hence *eight inches should usually be adopted as the minimum size for sanitary sewers.*

Usually the great bulk of the sanitary sewers in a separate system will be of this minimum size, only a limited length of the larger sizes being required for sub-mains and mains. See the sewerage map of Ames, Iowa, Fig. 38.

In the early use of the separate system, many 6-inch laterals were constructed, and, except for occasional stoppages from articles improperly put into the sewers, they have worked well. Some engineers still use six inches as the minimum size.

48. Minimum Grades and Velocities for Separate Sanitary Sewers. In the design and construction of sewers it has been found that certain minimum grades should be adopted to prevent deposits, no sewers being built to lighter grades than the minimum unless special means for flushing, or special facilities for cleaning, are provided. This is to insure sufficient velocity to prevent the settling out of the solid matter in the sewage to form deposits in the sewers.

These minimum grades for separate sanitary sewers are as follows:

TABLE II
Minimum Grades for Separate Sanitary Pipe Sewers

DIAMETER	MINIMUM GRADE	DIAMETER	MINIMUM GRADE
4 inches	1.20 per cent	18 inches	0.12 per cent
6 "	0.67 " "	20 "	0.10 " "
8 "	0 43 " "	24 "	0.08 " "
9 "	0.36 " "	27 "	0.07 " "
10 "	0.30 " "	30 "	0.06 " "
12 "	0.23 " "	33 "	0.05 " "
15 "	0.16 " "	36 "	0.045 " "

CAUTION.—*For the above minimum grades to be satisfactory and safe, there must be enough sewage to give a good depth of flow.*

The flow and velocity in a sewer fluctuate greatly, as illustrated in Article 52, below, the velocity at low flow being much less than when flowing full or half-full.

Experiments have shown that an actual velocity of $1\frac{1}{4}$ to $1\frac{1}{2}$ feet per second is sufficient to prevent deposits of the solid matters usually found in sanitary sewers; but to secure this velocity at low flow requires about 2 feet per second when the sewer flows full or half-full (see Figs. 30 and 31 for the fluctuation of velocity with depth of flow). Hence *the minimum grades for sanitary sewers should usually be those giving a velocity of* 2 *feet per second when flowing full or half-full, as shown by the diagrams, Figs.* 27, 28, *and* 29.

It is usually considered that, within a reasonable period in the future, the increased high-water flow each day should be sufficient to fill the sewer half-full or nearly so. However, in numerous cases, sanitary sewers have been observed to work well at the above grades with less depths of flow than this.

Much will depend on the nature of the sewage. Some thick, manufacturing sewages, heavily loaded with solid matter, would require considerably heavier grades to insure self-cleansing.

Where it is absolutely impossible to secure the above minimum grades, special means for flushing, such as automatic flush-tanks placed about three blocks apart, should be used.

49. General Explanation of the Calculation of Amount of Sanitary Sewage. The first thing necessary in computing the size required for any particular sanitary sewer, is to ascertain the amount of sewage it must carry. While this cannot be foretold with exactness, yet, by well-established methods, an approximation sufficiently close for all practical purposes can readily be made.

The *first step* in computing the amount of sewage will be to estimate the future tributary population which may use the sewer. For this, see Art. 50, below.

The *second step* will be to estimate the average amount of sewage contributed by each person per day—that is, the average flow of sewage per capita per day. This, multiplied by the tributary population, will give the total average amount of sewage per day which the sewer must carry.

Two methods are in use for estimating the average flow of sewage per capita per day:

(1) It is often assumed to equal the average consumption of water per capita per day. For this method, see Art. 51, below.

(2) The best method is to compare the local conditions with

actual sewer gaugings of flow in sewers under similar conditions elsewhere. For this method, see Art. 52, below.

50. Methods of Estimating the Population Tributary to Sanitary Sewers. The most important difficulty encountered in estimating the population tributary to sanitary sewers, is the fact that it is the *future* population which must be determined. To know the present tributary population is not sufficient. Two methods will be described:

(1) *The best method of estimating the future population tributary to sanitary sewers is as follows:*

(*a*) On the sewer map, lay out sewers to serve all districts to be served in the future as well as at present.

(*b*) After careful examination of the ground, and study of the conditions, estimate the number of persons tributary to the sewers per 100 feet of sewers in each district when it is built up as fully as can reasonably be expected.

In doing this, five or six persons per family should usually be allowed, and the number of families on both sides of the street for one block in the future estimated. The number of persons per block so obtained should then be divided by the number of hundred feet of sewer per block from center to center of streets.

Thus, if there are 6 lots 50 feet wide per block (=300 feet) on each side of the sewer, and the streets are 60 feet wide (=360 feet center to center of streets), and if it is thought that every lot will eventually contain one residence,

$$\text{Tributary population} = \frac{12 \times 6 \text{ persons}}{3.60} = 20 \text{ persons per 100 feet of sewer.}$$

The tributary population per 100 feet of sewer will usually range from 20 persons in the residence districts of small cities, to 100 persons in thickly built-up business districts. In the congested districts of the largest cities, the population is still denser.

(*c*) *To determine the total population tributary above any point on a sanitary sewer, scale from the sewer map the total number of hundred feet of tributary sewer above that point, including all branches; and multiply the total so obtained by the tributary population per* 100 *feet of sewer.*

Thus, if there are 8,500 ft. of tributary sewers, and the tributary population is 20 per 100 ft., the total tributary population will $= 156 \times 20 = 3{,}120$ persons. In some cases part of the length of tributary

sewers may have to be multiplied by one density of tributary population, and part by another.

(2) In case the future population of an entire city is to be estimated, a different method must be used.

Usually, the past population of the city at different dates is obtained from census reports; and by study of this past growth, and of the present and probable future local conditions as affecting growth, and by comparison with the past growth of larger cities whose conditions were similar, estimates are made of the probable future populations at different dates, for 20 to 50 years in the future.

Usually, also, the past records of the city that is being studied, and of others, are platted as curves on cross-section paper, the ordinates representing population, and the abscissæ dates; and the future estimates are made by prolonging the curve of growth into the future.

51. Use of Statistics of Water Consumption in Determining the Per Capita Flow of Sanitary Sewage. Since about 99.8 per cent of sanitary sewage is merely ordinary water, nearly always taken from the public supply, the total flow per capita of sanitary sewage is usually approximately equal to the consumption of water per capita (that is

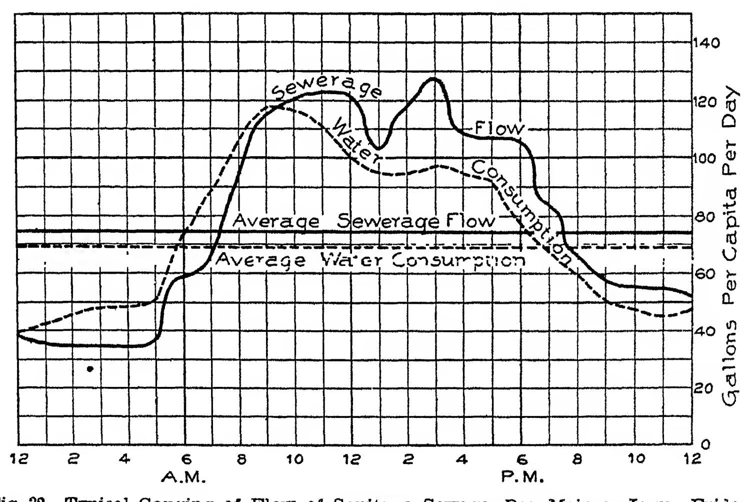


Fig. 32. Typical Gauging of Flow of Sanitary Sewage, Des Moines, Iowa, Friday July 5, 1895.

per person). In Fig. 32 may be seen how closely sewage flow and water consumption ordinarily correspond.

In many towns, however, there will not be such close correspondence. Sometimes considerable amounts of water may be used for manufacturing or other purposes which divert it from the sewers, making the sewage flow less than the water consumption. More often there will be considerable influxes of ground water through leaking sewer joints, sometimes making the sewage flow several times as great as the water consumption.

However, very extensive statistics of water consumption in a large number of places have been collected, while actual gaugings of flow of sewage are comparatively few. Hence statistics of the water consumption of the town for which sewers are being designed, or of similar towns elsewhere, are often used as the basis for estimating the per capita flow of sanitary sewage. In studying each town

TABLE III

Consumption of Water in American Cities, 1895

City	Population	Daily Consumption per Person, 1895. Gallons
New York	3,437,202	100
Chicago	1,698,575	139
Philadelphia	1,293,697	162
St. Louis	575,238	98
Boston	560,892	100
San Francisco	342,782	63
Buffalo	352,387	271
New Orleans	287,104	35
Minneapolis	202,718	88
Columbus	125,560	127
Atlanta	89,872	42
Nashville	80,865	139

preliminary to designing sewers for it, all possible information should be secured relative to its water consumption.

On pages 4 to 10 of the instruction paper on Water Supply, Part I, will be found a detailed discussion of water consumption. From a larger table given there, Table III herewith is condensed, to show how the average per capita water consumption varies in different American cities.

It will be noted that there is a very wide range in water consumption. The excessively low rates usually mean an incomplete water supply, which is likely to be extended later, while the excessively high rates usually mean great waste of water. This can often be greatly reduced by introducing water meters.

Under fairly average conditions the consumption will usually fall between the limits of 40 and 125 gallons per capita per day, as shown in detail in Table IV.

TABLE IV

Water Consumption under Ordinary Conditions

Use	Gallons per Capita per Day		
	Minimum	Average	Maximum
Domestic	15	25	40
Commercial	7	20	35
Public	3	5	10
Waste and Loss	15	25	40
Total	40	75	125

52. Use of Sewer Gaugings in Determining the Per Capita Flow of Sanitary Sewage. It has already been stated that the flow of sanitary sewage is not always equal to the water consumption. In one case of sewer gaugings, the writer found the flow of sewage to be only 50 to 60 per cent of the water consumption, the remainder of the water being consumed for purposes which diverted it from the sewers. In another case of sewer gaugings, the writer found the flow of sewage to be over 500 per cent of the water consumption, the increase being due to infiltration of ground water through sewage joints. Hence, water consumption data alone are not sufficient in making estimates of sewage flow, and data from actual sewer gaugings are needed. Of late years there is an increasing accumulation of data of sewage flow obtained from actual gaugings. Some of these data are given in Table V.

At the Iowa State College, the sewage flow, as given in Table V, below, was 50 to 60 per cent of the water consumption, owing to uses of water which diverted it from the sewers. At Grinnell, on the other hand, infiltration of ground water into the sewers increased the sewage flow to about six times the total water consumption on the same day.

A study of Table V will show, however, that *in general the average flow of sanitary sewage is between the limits of* 50 *and* 125 *gallons per capita per day.*

53. Capacities of Sanitary Sewers Required to Provide for Fluctuations in the Rate of Flow. So far our discussion of flow of

TABLE V
Gaugings of Flow of Sanitary Sewage

Sewer	Date	Duration, Days	Tributary Population	Sewage Flow, Gals. per Capita per Day		
				Min.	Av.	Max.
Compton Ave., St. Louis	1880	6	8, 200	65	102	149
College St., Burlington, Vt.	1880	5–8	325	65	115	140
Huron St., Milwaukee, Wis.	1880	–	3, 174	—	—	120
Memphis, Tenn.	1881	–	20, 000	61	—	140
13 Sewers, Providence, R. I.	1884	1–6	33, 825	—	78	—
Asylum, Binghamton, N. Y.	1888	–	1, 300	—	—	608
16 Sewers, Toronto, Ont.	1891	3	168, 081	—	87	—
Insane Asylum, Weston, W. Va.	1891	2	1, 000	40	91	151
Schenectady, N. Y.	1892	1	*10, 000	72	86	103
Canton, Ohio	1893	–	40, 000	54	129	180
Chautauqua, N. Y.	1894	–	7, 000	6	20	30
Iowa State College, Ames, Ia.	1894	7	289	0	32	77
Des Moines, Ia., E. Side	1895	15	8, 100	22.5	74	142
Des Moines, Ia., W. Side	1895	13	19, 400	23.2	66	175.3
Iowa State College, Ames, Ia.	1900	2	800	54	95	175
Iowa State College, Ames, Ia.	1900	28	800	30	57	130
Marshalltown, Ia.	1900	1	4, 200	67	85	111
Grinnell, Ia.	1901	1	2, 000	169	186	200
Insane Asylum, Mt. Pleasant, Ia.	1901	1	1,2 00	32	62	115
Waverly, N. Y.	1905	4	1, 796	79	155	194

* Estimated.

sanitary sewage (Arts. 51 and 52) has referred particularly to the *average* flow per capita per day. The flow, however, is not uniform, but fluctuates greatly. First, there is a *seasonal fluctuation.* The flow is apt to be especially high in severe cold weather, when faucets are left running to keep pipes from freezing; in hot weather, when water consumption is high; and in wet weather, when some ground water finds its way into the sewers.

Second, there is a *daily fluctuation.* For example, gaugings show that the flow usually is light on Sundays and holidays, when business is suspended. The flow on Monday is apt to be especially high, on account of wash day.

Third, there is an *hourly fluctuation,* at different times of the day and night. In Fig. 32, an example is shown of the fluctuation of sewage flow throughout one day, as determined by a continuous sewer gauging in the case of a city of 56,000 population. As shown in this figure, the flow of sanitary sewage is usually low through the night, reaching a minimum at about 2 to 3 A. M. It increases rapidly early in the morning, reaching a high point at about 10 to 11 A. M. Although there is usually a temporary drop at the noon

hour, the flow continues high until early evening, and then decreases rapidly to its low night value.

A study of the sewer gaugings summarized in Table IV, together with others, shows that the flow of sanitary sewage ordinarily fluctuates from a minimum rate of 30 per cent to a maximum rate of 265 per cent of the average rate. If the gaugings had been extended over longer periods of time, still greater fluctuations of flow would certainly have been found.

It is apparent that the fluctuations in rate of flow will be greater in lateral sewers than in main sewers. To make them large enough to provide for the greatest rates of flow to be reasonably expected, *sanitary sewers should be given the following capacities:*

PROPER CAPACITIES OF SANITARY SEWERS

For lateral sewers, 350 per cent of the average flow.
For sub-main sewers, 325 per cent of the average flow.
For main sewers, 300 per cent of the average flow.

Table VI (page 68) is proportioned on the above basis.

54. Ground Water in Sanitary Sewers. In addition to the sanitary sewage itself, provision must often be made in separate sanitary sewers for leakage of ground water into the sewers. The amount of ground water to be allowed for, will depend on the character of the soil, on the height of the ground water with reference to the sewer, and on the care with which the sewer joints are made. *If the joints are made very carefully, the amount of ground water to be expected may range, with the soil, and height of ground water, from 0 to 30,000 gallons per mile.* This will constitute, say, 0 to 30 per cent of the sewage, but is a steady flow, not requiring the 300 to 350 per cent allowance for fluctuations required for sewage (see Art. 53). Hence, *if the joints are carefully made, the capacity of the sewers need not be increased more than 10 per cent for ground water.*

If sub-drains with outlets separate from the sewers are provided for all wet stretches of trench, no allowance whatever for ground water need be made in the size of the sewers.

The infiltration of ground water is apt to be much greater during and immediately after the construction of sewers than later, for the effect of sewers is to lower permanently the level of the ground water.

55. Summary of Methods of Computing Sizes of Separate Sanitary Sewers. The methods for computing the sizes of sanitary sewers may be summarized as follows:

(1) Lay out on the sewer map all the sewers required to serve all districts which can reasonably be expected to be included in the system, either at present or within say 30 to 50 years in the future.

(2) By a careful study of the topography, business conditions, manufacturing possibilities, and other future prospects, together with the sizes of blocks and lots, and the widths of streets, determine the probable future tributary population in each district per 100 feet of sewer, allowing usually five or six persons per family.

(3) By a careful study of the statistics of water consumption (Art. 51), and by comparison with actual sewer gaugings (Art. 52), taking into account all local conditions, estimate the average flow of sewage in gallons per capita per day.

(4) Beginning at the upper ends of the sewers, scale from the map and tabulate the total lengths of tributary sewer above successive points in the system, to the outlet. Multiply the number of hundreds of feet in these lengths by the tributary population per 100 feet, and by the average per capita flow of sewage per day, to get the total flow of sanitary sewage at the successive points.

(5) To allow for fluctuations (Art. 53), multiply the above average rates of flow of sanitary sewage by

$3\frac{1}{2}$ for lateral sewers;
$3\frac{1}{4}$ " sub-main "
3 " main "

to get the maximum rates of flow of sanitary sewage.

(6) To the maximum rates of flow so found, add 0 to 30,000 gallons per mile of tributary sewers, to allow for ground water (Art. 54).

(7) Occasionally it may be necessary also, in the case of certain sewers, to make special allowances for manufacturing sewage from large factories, each factory being studied by itself to determine its probable sewage flow. This flow will usually be subject to as much fluctuation as sanitary sewage, and hence must be multiplied by the factors given in 5, above.

(8) On the sewer profiles (see Art. 92), the grades of the sewers at the successive points will be determined and shown. Using these grades, and the total maximum rates of flow of sewage determined

TABLE VI
Sizes Required for Separate Sanitary Pipe Sewers

1	2	3	4	5	6	7	8	9
Diam of Sewer, Ins.	Grade of Sewer, %	Maximum Permissible Av. Flow, Gals. per Day	Maximum Permissible Tributary Population			Maximum Permissible Linear Feet of Tributary Sewer for 20 Persons per 100 Feet		
			Gals. per Capita per Day			Gals. per Capita per Day		
			75	100	125	75	100	125
8	0.43	130,000	1,700	1,300	1,000	8,700	6,500	5,200
	0.60	160,000	2,100	1,600	1,300	11,000	8,000	6,400
	0.80	180,000	2,400	1,800	1,400	12,000	9,000	7,200
	1.00	200,000	2,700	2,000	1,600	13,000	10,000	8,000
	1.40	240,000	3,200	2,400	1,900	16,000	12,000	9,600
10	0.30	220,000	2,900	2,200	1,800	15,000	11,000	8,800
	0.40	260,000	3,400	2,600	2,100	17,000	13,000	10,000
	0.60	310,000	4,100	3,100	2,500	21,000	15,000	12,000
	0.80	360,000	4,800	3,600	2,900	24,000	18,000	14,000
	1.00	400,000	5,300	4,000	3,200	27,000	20,000	16,000
12	0.23	350,000	4,700	3,500	2,800	23,000	17,000	14,000
	0.40	460,000	6,100	4,600	3,700	31,000	23,000	18,000
	0.60	560,000	7,500	5,600	4,500	37,000	28,000	22,000
	0.80	650,000	8,700	6,500	5,200	43,000	32,000	26,000
	1.00	720,000	9,600	7,200	5,800	48,000	36,000	29,000
15	0.17	550,000	7,300	5,500	4,400	37,000	27,000	22,000
	0.30	750,000	10,000	7,500	6,000	50,000	37,000	30,000
	0.40	850,000	11,000	8,600	6,900	57,000	43,000	34,000
	0.60	1,000,000	13,000	10,000	8,000	67,000	50,000	40,000
	0.80	1,200,000	16,000	12,000	9,600	80,000	60,000	48,000
18	0.13	800,000	11,000	8,000	6,400	55,000	40,000	32,000
	0.30	1,200,000	16,000	12,000	9,600	80,000	60,000	48,000
	0.40	1,400,000	19,000	14,000	11,000	93,000	70,000	56,000
	0.60	1,700,000	23,000	17,000	14,000	113,000	85,000	68,000
	0.80	2,000,000	27,000	20,000	16,000	134,000	100,000	80,000
24	0.10	950,000	12,000	9,500	7,400	62,000	46,000	37,000
	0.20	1,300,000	17,000	13,000	10,000	87,000	65,000	52,000
	0.40	1,900,000	25,000	19,000	15,000	126,000	95,000	76,000
	0.60	2,300,000	31,000	23,000	18,000	153,000	115,000	92,000
	0.80	2,600,000	35,000	26,000	21,000	173,000	130,000	104,000
27	0.08	1,400,000	19,000	14,000	11,000	93,000	70,000	56,000
	0.20	2,200,000	29,000	22,000	18,000	147,000	110,000	88,000
	0.30	2,700,000	36,000	27,000	22,000	180,000	135,000	108,000
	0.40	3,100,000	41,000	31,000	25,000	207,000	155,000	124,000
	0.60	3,800,000	51,000	38,000	30,000	254,000	190,000	152,000
30	0.06	2,200,000	29,000	22,000	18,000	147,000	110,000	88,000
	0.10	2,800,000	37,000	28,000	22,000	187,000	140,000	112,000
	0.20	4,000,000	53,000	40,000	32,000	267,000	200,000	160,000
	0.40	5,700,000	76,000	57,000	46,000	380,000	285,000	238,000
	0.60	7,000,000	93,000	70,000	56,000	466,000	350,000	280,000
36	0.05	3,200,000	43,000	32,000	26,000	214,000	160,000	128,000
	0.10	4,600,000	61,000	46,000	37,000	307,000	230,000	184,000
	0.20	6,500,000	87,000	65,000	52,000	433,000	325,000	260,000
	0.40	9,300,000	124,000	93,000	74,000	620,000	465,000	372,000
	0.60	11,400,000	152,000	114,000	91,000	760,000	570,000	456,000

in 5, 6, and 7, above, refer to Fig. 27 for pipe sewers, or to Fig. 28 for brick or concrete sewers, and find the sizes of sewers required.

Example 28. In a town in which the blocks are 340 feet, center to center of streets, there are 14 lots per block. The total length of tributary sewers above a certain point on a sub-main sewer in the system (separate sewers), is 16,600. The conditions affecting rate of sewage flow per capita are average. No allowance need be made for ground water or manufacturing sewage. The grade of the sewer is 0.30 per cent. What size is required?

Solution. The tributary population will be $\frac{14 \times 6}{3.4} = 25$ persons per 100 feet of sewer. The average rate of flow may be assumed at 85 gallons per capita per day. Hence the maximum rate of flow for this sub-main sewer will be $166 \times 25 \times 85 \times 3\frac{1}{4} = 1{,}150{,}000$ gallons per day.

Hence, by Fig. 27, for a 0.30 per cent grade, a 12-inch pipe sewer will be required.

Answer. A 12-inch pipe sewer.

56. Table of Sizes Required for Sanitary Sewers. By the methods given in Art. 55, omitting allowances for ground water and manufacturing sewage, Table VI (page 68) has been computed, to reduce the labor of computation of sizes of separate sanitary pipe sewers.

TO USE THE TABLE

Proceed to follow out steps 1, 2, 3, and 4, in Art. 55, just above (which read), thus determining the total estimated future number of linear feet of tributary sewer at successive points, the estimated future number of persons tributary per 100 feet of sewer (which let $= P$), and the estimated average flow of sewage in gallons per capita per day (which let $= F$). Also ascertain the grade to which the sewer is to be built.

(*A*) *If $P = 20$ persons per* 100 *feet, and if F lies between* 75 *and* 125 *gallons per capita per day, and if no allowance is necessary for ground water or manufacturing sewage,* find in column 7, 8, or 9, or by interpolating between them, according to the value of F, a number close to the calculated number of linear feet of tributary sewer opposite to the given sewer grade, interpolating between the grades, and take the corresponding size of sewer in column 1.

Example 29. For 13,100 linear feet of sewer, 20 persons per 100 ft., 85 gallons per capita per day, and 0.35 per cent grade.

We find that for a 0.35 per cent grade an 8-inch sewer would be considerably too small, as shown by interpolating between the numbers in columns 7 and 8, while a 10-inch sewer would be a little larger than needed.

Answer. A 10-inch pipe sewer.

(*B*) If P does not $= 20$ persons per 100 feet (the other conditions remaining as in A, above), first multiply the number of linear feet of tributary sewer by $\frac{P}{20}$, and then proceed as in A, just above.

Example 30. For 16,300 linear feet of sewer, 30 persons per 100 feet of sewer, 110 gallons per capita per day, and a sewer grade of 0.25 per cent.

We first find $16{,}300 \times \frac{30}{20} = 24{,}450$ linear feet. Then interpolating between columns 8 and 9, we find that for a 0.25 per cent grade a 12-inch would be considerably too small, while a 15-inch sewer is a little larger than needed.

Answer. A 15-inch pipe sewer.

(*C*) If F (rate of sewage flow) is less than 75 or more than 125 gallons per capita per day, first multiply the number of linear feet of tributary sewer by $\frac{F}{100}$, and then by $\frac{P}{20}$ (where P = persons per 100 feet of sewer), and then find the nearest number in column 8 opposite the given grade.

Example 31. For 22,500 linear feet of sewer, 35 persons per 100 feet, 150 gallons per capita per day, and 0.45 per cent grade.

We first find $22{,}500 \times \frac{150}{100} \times \frac{35}{20} = 59{,}000$ linear feet. In column 8 we find that for a 0.45 per cent grade a 15-inch sewer would be considerably too small, while an 18-inch is too large.

Answer. An 18-inch pipe sewer.

(*D*) *If ground water or manufacturing sewage, or both*, must be allowed for, ascertain the *total average sewage flow*, by multiplying the linear feet of tributary sewer by $\frac{P}{100}$ (P = persons per 100 feet), and this result by F (= gallons per capita per day, of sanitary sewage), and by then adding to this result the total allowance for manufacturing sewage, and $\frac{1}{3}$ the total allowance for ground water. Then find by interpolation in column 3 the nearest number opposite the given grade, and take the corresponding size of sewer.

Example 32. For 15,600 linear feet of tributary sewer, 25 persons per 100 feet, 85 gallons per capita per day, 15,000 gallons per day per mile ground water, 200,000 gallons per day manufacturing sewage, and 0.20 per cent grade

We find the total average flow of sewage to use is $15{,}600 \times \frac{25}{100} \times 85 + 200{,}000 + \frac{15{,}000}{3} \times 3$ (miles) $= 546{,}000$ gallons per day. In column 3 we find that for a 0.20 per cent grade, a 12-inch sewer would be considerably too small, while a 15-inch is a little larger than is needed.

Answer. A 15-inch pipe sewer.

GENERAL EXAMPLE FOR PRACTICE IN DESIGNING SEPARATE SANITARY SEWERS

57. Working out the following example will materially help the student.

Example 33. Calculate the size of the outlet sewer of the sewer system shown in Fig. 4, assuming that there will be in the future 20 persons tributary per 100 feet of sewer, that the average flow of sewage will be 100 gallons per capita per day, no special allowance for ground water or manufacturing sewage being needed. Also assume that there may be in the future 15,000 feet of sewer extensions not shown in the figure. The grade of the outlet sewer is 0.20 per cent. Assume scale of drawing, 1,500 feet per inch

Solution. Take a long strip of paper with one edge straight; and on this, mark off with a pencil a scale of feet from the scale assumed above. With this, scale off the lengths of all the sewers shown, except the storm sewers. Add up the lengths scaled, and add 15,000 linear feet of future extensions, to get the total length of tributary sewer. Then use Table VI.

Answer. An 18-inch pipe sewer.

CALCULATION OF SIZES AND MINIMUM GRADES OF STORM AND COMBINED SEWERS

58. Storm and Combined Sewers Calculated by Same Methods. In combined sewers the rate of flow of sanitary sewage is so small in time of storms in proportion to that of the storm sewage, that the sanitary sewage can be neglected altogether in calculating the size. For example, a combined sewer one mile long, with 20 persons tributary per 100 feet, and 75 gallons per capita per day, would have a maximum rate of flow of sanitary sewage at its lower end of $\frac{52.8 \times 20 \times 75 \times 3\frac{1}{2}}{7\frac{1}{2} \times 86{,}400} = 0.43$ cu. ft. per second (there being $7\frac{1}{2}$ gals. in 1 cu. ft., and 86,400 seconds in 1 day, and the maximum rate of flow being $3\frac{1}{2}$ times the average).

If the blocks are 360 feet wide, center to center of streets, this same sewer would have to take the storm sewage from $43\frac{1}{2}$ acres. The amount of this at the time of the maximum storm allowed for, calculated by the methods described below, would probably be at least 20 cu. ft. per second. The sanitary sewage would therefore be only about 2 per cent of the storm sewage. The amount of the latter cannot be foretold nearly so close as 2 per cent. Thus the sanitary sewage would have no appreciable effect upon the size of the combined sewer, and can be neglected.

59. Minimum Sizes of Storm and Combined Sewers. In the case of sanitary sewers, 8 inches was stated to be the minimum allowable diameter (see Art. 47); but in the case of sewers carrying storm sewage, there is much greater danger of stoppages from dirt, sticks, and other debris washed in from the surface during storms. Hence *twelve inches should be the minimum allowable diameter for storm and combined sewers.*

60. Minimum Grades and Velocities for Storm and Combined Sewers. It was stated in connection with sanitary sewers (Art. 48), that the minimum allowable velocities to prevent deposits should be

TABLE VII

Minimum Grades for Storm and Combined Sewers

Shape	Material	Size	Minimum Grades to Give Velocities of 3 ft. per Sec.	Minimum Grades to Give Velocities of 4 ft. per Sec.
Circular	Pipe	12-in. Diam.	0.48	0.88
"	"	15 "	0.34	0.62
"	"	18 "	0.25	0.47
"	"	24 "	0.17	0.31
"	"	30 "	0.13	0.23
"	Brick or Concrete	3-ft. "	0.14	0.25
"	"	4 "	0.10	0.17
"	"	5 "	0.07	0.12
"	"	6 "	0.06	0.10
"	"	7 "	0.05	0.08
"	"	8 "	0.04	0.06
"	"	9 "	0.03	0.05
"	"	10 "	0.025	0.045
Egg-Shaped	"	2 ft. × 3 ft.	0.20	0.35
"	"	$2\frac{1}{2}$ " × $3\frac{3}{4}$ "	0.15	0.26
"	"	3 " × $4\frac{1}{2}$ "	0.12	0.20
"	"	4 " × 6 "	0.08	0.14
"	"	5 " × $7\frac{1}{2}$ "	0 06	0.10
"	"	6 " × 9 "	0.05	0.08
"	"	7 " × $10\frac{1}{2}$ "	0.04	0.07

$1\frac{1}{4}$ feet per second at the minimum depths of flow, which will require grades sufficient to give minimum velocities of 2 feet per second when the sewer flows full or half-full. For sewers carrying storm sewage, however, greater minimum velocities are necessary to prevent deposits, on account of the dirt, pebbles, and other heavy rubbish washed into them from the surface in times of storms. *For combined and storm sewers the minimum allowable grades should be steep enough to give a minimum velocity of 3 feet per second. If practicable without too great expense, 4 feet per second should be secured.*

61. General Explanation of the Calculation of Amount of Storm Sewage. When rain begins to fall upon the area drained by a storm sewer, the water falling in the immediate neighborhood of the outlet at once enters the sewer and begins to be discharged. As time passes and the rain continues, water arrives at the outlet from more and more remote portions of the drainage area, and the discharge at the outlet increases quite rapidly until water is being discharged from all portions of the drainage area at the same time. After that, any further increase is slow, being due only to a per cent of run-off slowly increasing as the saturation of the soil becomes more complete.

The *time of concentration* is the longest time required for water from the remotest points of the portion of the drainage area being considered, to reach the outlet of that portion.

The *general law of the heaviest rainfalls,* the ones which determine the sizes of sewers, is that the heaviest rates for short storms are much greater than the heaviest rates for long storms. *The longer the time, the less will be the average rate of the maximum storm lasting that time.*

In Fig. 33, is given a diagram prepared by Prof. A. N. Talbot, showing rainstorms in the Central States. On this diagram the ordinates represent the rate of rainfall in inches per hour (which = cu. ft. per second per acre), while the abscissæ represent the duration of the storm. Three curves are also shown, one for very rare rainfalls, one for ordinary heavy rains, and one intermediate. On the diagram each + represents o e storm.

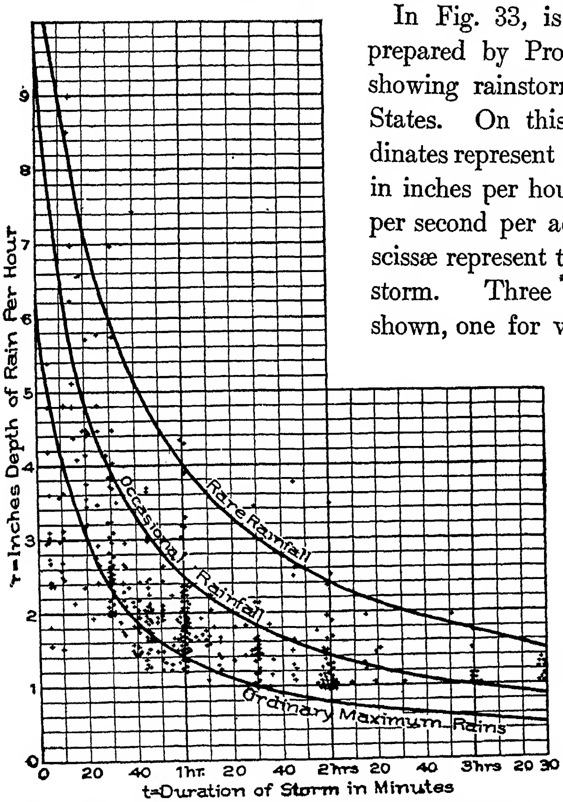


Fig. 33. Rates of Heavy Rainfall in the North Central States, Ohio, Indiana, Illinois, Missouri, Kansas, and Iowa.

The storm causing the greatest rate of discharge in a storm sewer will usually be the maximum rain lasting a length of time equal to the time of concentration. If a time less than this be taken, water will not be discharged at the outlet from all parts of the drainage area at once, and that from near the outlet will have a chance to run away before that from the remotest points arrives. On the other hand, if a time be taken longer than the time of concentration, the *heaviest* rate of the maximum storm lasting this long will be less

than the rate of the maximum storm lasting a length of time just equal to the time of concentration; and since the storm is lighter the flow will be lighter.

Not all of the water falling on a drainage area will be carried away in the sewer. During and after the storm, some of the water is evaporated into the air, and some is absorbed into the soil. Some also accumulates on the surface, to flow off into the sewer after the rain has ended. *The engineer determines the percentage of the rain flowing off in the sewer, by estimating the percentage of maximum run-off of the drainage area.*

The general method for calculating the amount of storm sewage for any particular drainage area, is therefore as follows:

(*a*) Calculate the *time of concentration,* or longest time of flow to the point for which the size of sewer is being determined.

(*b*) Calculate the *rate of maximum rainfall* corresponding to the time of concentration.

(*c*) Calculate the *percentages of impervious and pervious areas* on the watershed drained by the sewer.

(*d*) Using the percentages of impervious and pervious areas obtained in *c,* calculate the *maximum percentage of run-off,* or the percentage of the rate of the maximum rainfall which will be running off in the sewer under design at the end of the time of concentration.

(*e*) *Calculate the total maximum rate of flow of storm sewage,* by *multiplying together* the *drainage area,* the *maximum rate of rainfall* corresponding to the time of concentration, and the *maximum percentage of run-off.*

62. Calculation of the Time of Concentration. The time of concentration, which is the longest time required for water falling on the remote portions of the watershed to flow to the point for which the size of sewer is being determined, will be the sum of, (1), the time required for the water from roofs, yards, sidewalks, and pavements to reach the sewers by way of the gutter and street inlets, and, (2), the longest time required for the water to flow through a line of sewers to the point for which the size of sewer is being calculated.

(1) *Time Required for Water from Roofs, Gutters, etc., to Reach the Sewers.* This will usually be between the limits of 5 and 15 minutes, depending on the steepness of the slopes of the surface and of the gutters, on the distance the water must flow to reach the gutters and the distance it must flow in the gutters to reach the street inlets, on the character of the surface (whether it offers obstructions to flow or not), or whether the roofs are connected to the gutters or directly to

the sewers, etc. By looking over the ground carefully, and allowing for the above conditions in a general way, the time may be estimated as closely as the data will warrant, without special calculations. The upper limit of 15 minutes may be used when the gutters have a very light grade, and are two blocks long, and where the roofs discharge into the gutters instead of into the sewer direct.

(2) *Longest Time Required for the Water to Flow through the Sewers.* This is computed by taking the grades and sizes of the different parts of usually the longest line of sewers, and determining the corresponding velocities of flow by the use of the sewer diagrams, Figs. 27, 28, and 29, already given. From these velocities, and the lengths of the several portions of the sewer, the corresponding times required for the sewage to flow through each part can be readily computed, and their sum will be the time required. The designing must be begun at the upper ends of the sewers, so that we may know the sizes of sewer needed in computing the times of flow through each portion.

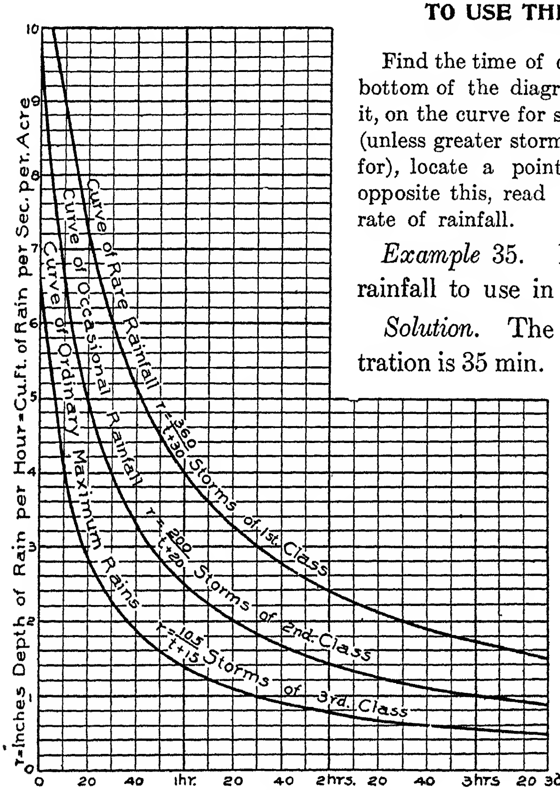
Example 34. Required the time of concentration in the following case: The longest sewer consists of 400 feet of 18-inch pipe sewer, grade 0.5 per cent; 800 ft. of 24-inch pipe, grade 0.3 per cent; 1,200 ft. of 36-inch brick sewer, grade 0.25 per cent; 2,400 ft. of 48-inch brick sewer, grade 0.17 per cent. The roofs discharge into the gutters, through which the sewage must flow 2 blocks at 0.5 per cent grade to reach a street inlet.

Solution:

	Velocity	Time
Estimated for water from roofs and gutter to reach sewer		15.0 min.
In 18-inch sewer, Fig. 27	4.2 ft. per sec.	1.6 "
" 24-inch " " 27	4.0 " " "	3.3 "
" 36-inch " " 28	4.0 " " "	5.0 "
" 48-inch " " 28	4.0 " " "	10.0 "
Answer. Total time of concentration =		35 "

63. Calculation of the Rate of Rainfall Corresponding to the Time of Concentration. In Fig. 34 are reproduced separately the three rainfall curves shown in Fig. 33. Storms of the 1st and 2d classes are rare, and are so very heavy that it would be excessively expensive to build sewers large enough for them. Hence sewers are usually built only large enough to provide for storms of the 3d class.

It is considered less expensive to suffer some damage from rare overcharging of the sewers than to build the greater sizes, though in case very valuable property would be damaged it may be wiser to provide for the heaviest storms.



t=Duration of storm in minutes=Time of concentration=Time required for water to flow from the remotest part of the area drained to the point under consideration on the sewer.

Fig. 34. Diagram Showing Rates of Maximum Rainfall to be Used in Calculating the Size of Storm Sewers.

TO USE THE DIAGRAM

Find the time of concentration at the bottom of the diagram. Vertically over it, on the curve for storms of the 3d class (unless greater storms are to be provided for), locate a point; and horizontally opposite this, read off on the left the rate of rainfall.

Example 35. Find the rate of rainfall to use in example 34.

Solution. The time of concentration is 35 min. Over this we read on the curve for 3d-class storms, 2.1 inches per hour.

Answer. 2.1 inches per hour.

64. Calculation of the Percentages of Impervious and Pervious Areas on the Sewer Watershed. The percentage of impervious area may be calculated in the following manner:

Take a typical unit of area, usually one average block, and divide it into different classes of surfaces, having different percentages of imperviousness, as follows:

(*a*) *Roof Area.* From the average size of buildings, and the average number of buildings per block which will be connected with

the sewers or with the gutters, calculate the total roof area in the block. Take this at its full value if the roofs are connected directly with the sewers, but take only 90 per cent if the roofs are connected with the gutters.

(*b*) *First-Class Pavements.* Calculate the total area, per block, of brick, asphalt, stone block, and similar first-class pavements, with tight joints, and take 80 per cent of this area.

(*c*) *Second-Class Pavements.* Calculate the total average area per block, and take 60 per cent.

(*d*) *Third-Class Pavements.* Calculate the total average area per block of good macadam and similar pavements, and take 40 per cent.

(*e*) *Hard-Earth Roads.* Calculate the total average area per block of the traveled, hard-earth surfaces, and take 20 per cent.

(*f*) *Sidewalks.* Calculate the several total average areas per block of 1st, 2d, and 3d-class sidewalks, corresponding to the classes of pavements in *b*, *c*, and *d*, above. If these extend to the gutters, as in business districts, take the same percentages as for the corresponding classes of pavements—namely, 80, 60, and 40 per cent for 1st, 2d, and 3d-class sidewalks, respectively. But if the pavements are separated from the gutters by wide parking, as in the residence districts, take only one-half the above percentages—namely, take 40, 30, and 20 per cent, for 1st, 2d, and 3d-class sidewalks, respectively.

Finally, add together all the reduced average areas per block (a, b, c, d, e, and f) obtained as above explained, and divide the sum by the total area of the typical block. The quotient will give the percentage of impervious area.

The percentage of pervious area is obtained by subtracting the percentage of impervious area from 100 *per cent.*

Example 36. In examples 34 and 35, assume the typical block to be 360 ft. square, center to center of streets, as follows:

Streets, 60 ft. wide; pavements, 30 ft. wide; asphalt on two streets; good macadam on the other two; cement sidewalks, 5 ft. wide, on all four streets.

One alley 20 ft. wide.

Lots, 12 in number, each 50 × 140 ft., each lot containing one house, the houses averaging 30 × 40 ft., the roofs connected with the gutter.

Calculate the percentage of impervious and pervious area.

Solution:

(*a*)	Roofs,	$30 \times 40 \times 12 \times .90 =$	12,960 sq. ft.
(*b*)	1st-Class Pavements,	$2 \times 15 \times 360 \times .80 =$	8,640 "
(*d*)	3d-Class Pavements,	$2 \times 15 \times 330 \times .40 =$	3,960 "
(*f*)	1st-Class Sidewalks,	$5 \times 1{,}210 \times .40 =$	2,420 "
	Total impervious area per block	$=$	27,980 sq. ft.

Total area of one block $= 360 \times 360 = 129{,}600$ sq. ft.

Answer. *Percentage of impervious area* $= \frac{27{,}980}{129{,}600} = 21.58$ per ct.

Percentage of pervious area $= 100 - 21.58 = 78.42$ per cent.

Mr. Emil Kuichling, M. Am. Soc. C. E., has calculated the percentages of impervious area in various cities of New York State, and his work has been repeated by Prof. H. N. Ogden,* who finds the percentage to vary with the intensity of population, as follows:

TABLE VIII

Approximate Percentages of Impervious Area in Cities

Population per Acre	Percentage of Impervious Area	Percentage of Pervious Area
5	4	96
10	9½	90½
15	15	85
20	20½	79½
25	26	74
30	31½	68½
35	37	63
40	42½	57½
45	47½	52½
50	52½	47½
55	58	42

Even very heavily populated sections in the largest cities will seldom have more than 80 to 85 per cent of impervious area.

Table VIII furnishes an easy method of making approximate estimates of the percentages of impervious area.

Example 37. In example 36, estimate the percentage of impervious area by Table VIII.

Solution. The typical block contains 129,600 sq. ft.; and $\frac{129{,}600 \text{ (sq. ft.)}}{43{,}560 \text{ (sq. ft.)}} = 3$ acres. The 12 houses at an average of 5½ persons per house, would give 66 persons per block = 22 per acre.

* *Sewer Design*, p. 62.

Referring to Table VIII we find by interpolating, $22\frac{3}{4}$ per cent of impervious area, as compared with 21.6 per cent obtained above by the more exact method.

65. Calculation of the Maximum Percentage of Run-Off. Not all of the rain falling on the impervious area of a watershed will run off during the storm. Small amounts are evaporated or absorbed at once, for no city surfaces are absolutely impervious. A larger amount goes to fill up small depressions in the surfaces. A still larger amount accumulates on the surfaces of the watershed, making its way toward the sewer, the amount so accumulated and its rate of movement increasing as the storm continues at the same rate, until finally an equilibrium of flow is established, and the rate of the run-off from the impervious area becomes practically 100 per cent of the rainfall. Thus, the shorter the storm, the less the percentage of run-off from the impervious area; and hence sewer watersheds having the smallest times of concentration are likely to have the smallest percentages of maximum run-off from the impervious areas.

The maximum downpours which determine the size of the sewer, are often preceded by lighter downpours which saturate and partially flood the watershed. Hence *it will probably never be allowable to assume less than* 75 *per cent as the percentage of maximum run-off from the impervious areas of a sewer watershed, even with very short times of concentration, and comparatively little damage from overcharged sewers.*

With long times of concentration (say 45 minutes or more), and wherever great damage would be caused by overcharged sewers, 100 per cent of maximum run-off from the impervious areas should be assumed.

In the case of long-continued storms, the pervious area becomes gradually saturated, until some run-off occurs from it also. In the case of storms lasting several hours, such as cause the great floods in rivers, this percentage of maximum run-off may be quite high; but for sewers, the times of concentration, and hence the duration of the maximum downpour, are comparatively short—rarely as long as one hour.

For soils of average porosity and for moderate slopes, the percentage of maximum run-off from the pervious areas may be assumed to range from 0, *for* 15 *minutes time of concentration, to, say,* 20 *for* 1

hour's time of concentration. For porous, sandy soils and flat slopes, assume 0 *to* 50 *per cent, and for very impervious soils and very steep slopes,* 125 *to* 150 *per cent of the above percentages of maximum run-offs from pervious areas.*

Example 38. In examples 36 and 37, assume that the territory is a residence district, with moderate slopes and clay subsoil. Estimate the percentage of maximum run-off.

Solution. Since the time of concentration is only 35 minutes, while the damage from overcharged sewers would not be so great as in a business district, we shall assume 90 per cent maximum rate of run-off from the impervious area. For the pervious area, we interpolate roughly between 0 per cent for 15 minutes, and 17 per cent for 1 hour, and assume 8 per cent maximum rate of run-off.

$.90 \times 21.6$ per cent = 19.4 per cent from impervious area.
$.08 \times 78.4$ " " = 6.3 " " " pervious "

Answer. Total = 26 per cent maximum rate of run-off.

66. Summary of Methods of Computing Sizes of Storm Sewers. We may now summarize the methods of computing the sizes of storm sewers, described above in Articles 61 to 65, inclusive, as follows:

(*a*) Calculate the *time of concentration* (Art. 62), or longest time of flow from the remote portions of the sewer watershed to the point for which the size of sewer is being calculated.

(*b*) Calculate the *maximum rate of rainfall* (Art. 63) corresponding to the time of concentration.

(*c*) Calculate the *percentages of impervious and pervious areas* on the sewer watershed (Art. 64).

(*d*) From the percentages of impervious and pervious areas, and knowledge of the characteristics of the sewer watershed, calculate the *percentage of maximum run-off* (Art. 65).

(*e*) Calculate the *maximum rate of flow of storm sewage,* by *multiplying together* the *area of the sewer watershed in acres,* the *maximum rate of rainfall in inches per hour* (*b*), and the *percentage of maximum run-off* (*d*). The product will be the *cubic feet per second of maximum storm sewage flow.*

(*f*) Knowing the grade of the sewer, refer to Fig. 27, or Fig. 28, or Fig. 29, according to the shape and material of the sewer, and determine the size of sewer required to carry the maximum flow of storm sewage (*e*) when flowing full.

Example 39. In examples 34 to 38, assume that the sewer watershed is 5,280 feet long by 800 feet wide, and that the grade of the circular brick outlet sewer is to be 0.15 per cent. Calculate the required diameter.

(*a*) The time of concentration = 35 min. (see Ex. 34).
(*b*) The rate of maximum rainfall = 2.1 in. per hr. (see Ex. 35).
(*d*) The percentage of maximum run-off = 26 (see Ex. 38).
(*e*) The drainage area $= \frac{5{,}280 \times 800}{43{,}560} = 97$ acres.

$$97 \times 2.1 \times .26 = 53 \text{ cu. ft. per sec.}$$
$$= \text{maximum flow of storm sewage.}$$

(*f*) Referring to Fig. 28, we find, by interpolating between the 4-toot and 5-foot diameters, that for a grade of 0.15 per cent a diameter of 4 ft. 3 in. will be required for a circular brick sewer which can carry 53 cu. ft. per sec.

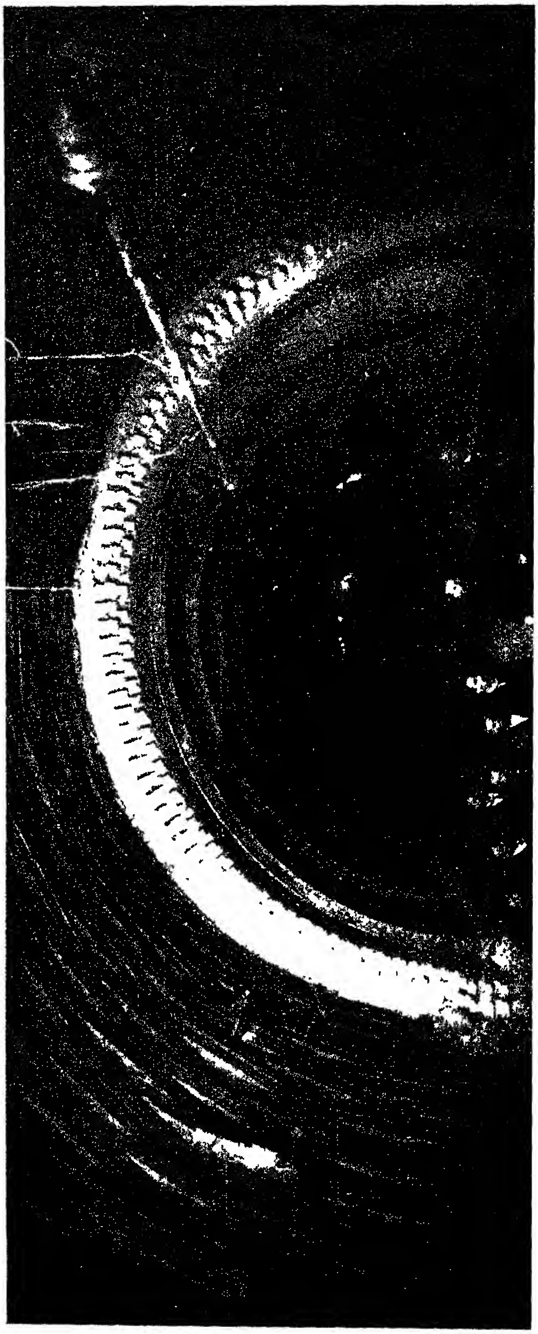
Answer. A 4 ft. 3 in. circular brick sewer.

GENERAL EXAMPLE FOR PRACTICE

67. Before proceeding further, the student should work out the following example in computation of the proper size of sewer:

Example 40. A thickly built-up sewer district, having a population of 35 persons per acre, : ntains 160 acres. The slopes are very flat, and the soil is sandy and porous. The longest line of sewers is 6,000 feet; and the velocity of flow in the sewers averages four feet per second. The roofs are connected with the gutters, in which the longest flow is two blocks. Calculate the diameter of the circular, brick outlet sewer, laid to a 0.08 per cent grade (NOTE: Use Table VIII.)

Answer. A 6-foot circular brick sewer.



VIEW OF 39th STREET INTERCEPTING SEWER, CHICAGO, ILL., THE LARGEST SEWER IN THE UNITED STATES

Courtesy of Meacham & Wright, Chicago, Ill.

SEWERS AND DRAINS

PART II

LAND DRAINS AND SUBDRAINS

68. General Discussion of Land Drains. Definitions of sewers and drains were given in Art. 1. Land drains have for their object the reclaiming of wet lands, to render them suitable for cultivation. The reclamation of wet lands also greatly improves the sanitary condition of the vicinity.

There are two principal kinds of land drains—namely, *tile drains*, or lines of agricultural drain tiles laid a few feet beneath the surface of the ground, to remove ground water; and *drainage ditches*, or open channels, made to serve as outlets for the tile drains and to drain ponds and remove surface water.

69. Planning and Construction of Land-Drainage Systems. When a tile drainage system is projected, a competent drainage engineer should at once be engaged to do the necessary surveying, plan the system, and pass on the construction.

The surveying will include the obtaining of data for a complete map of the system; and each drain should be staked out, stakes being set 50 feet apart, and an elevation taken with a good level at each stake. All the work should be checked.

The engineer should then prepare for the landowner a complete map of the system, to a scale of 200 to 400 feet per inch; also a sheet of profiles, including a profile of each drain, showing the depth and grade at all points. Without such map and profiles, knowledge of the system may be lost, and, on some future occasion, when very badly needed, may be unavailable.

The engineer should plan as simple and regular a tile system as possible, adopting long, parallel, straight lines of tile when practicable, with as few junctions as possible.

The grades may be very light in case of necessity, and short tile drains have worked well even at level grades; but the lighter the grade, the greater should be the care used in construction.

The minimum depths should usually be $3\frac{1}{2}$ to 4 feet. Shallower depths do not drain out the soil so thoroughly; and tile, if laid $3\frac{1}{2}$ to 4 feet deep, can be placed farther enough apart to more than make up for the cost of the greater depth.

The lines of tile should usually be placed from five to ten rods apart, depending on the soil—farthest apart in the most porous soil. The outlet should be built with special care; and a masonry wall should be constructed to hold the last length of tile.

For drainage ditches, careful surveys of the entire watershed must be made by a very competent engineer; and fully detailed plans and specifications must be prepared.

70. Contracts and Specifications for Tile Drains. The employer and the tile ditcher should sign a printed contract with detailed specifications, such as given herewith:

CONTRACT

It is hereby agreed between ...., employer, and ...., contractor, that the contractor shall, except for the furnishing of the tile along the ditch and the refilling of the ditch, entirely construct for the employer the following described drains:

....
....
....
....
....
....

It is further agreed that for the above work the employer shall pay the following prices:

....
....
....

It is further agreed that the employer............................ ....furnish board free to the contractor and his helpers during active prosecution of the work.

It is further agreed that the contractor shall begin the work by........ ....and complete the same by..................

It is further agreed that all the above work and the payments therefor shall be in strict accordance with the specifications given below and with the engineer's maps, profiles, and plans, all of which are hereby made a part of this contract.

Witness the hands of the respective parties, this................day of ....A. D.....

......................Employer
......................Contractor

SPECIFICATIONS

1. *Staking Out the Work.* The work will be staked out by the engineer, and his stakes must be carefully preserved and followed.

2. *Digging the Ditches.* The digging of each ditch must begin at its outlet, or at its junction with another tile drain, and proceed toward its upper end. The ditch must be dug along one side of the line of survey stakes, and about ten inches distant from it, in a straight and neat manner, and the top soil thrown on one side of the ditch and the clay on the other. When a change in the direction of ditch is made, it must be kept near enough to the stakes so that they can be used in grading the bottom. In taking out the last draft, the blade of the spade must not go deeper than the proposed grade line or bed upon which the tiles rest.

3. *Grading the Bottom.* The ditch must be dug accurately and truly to grade at the depths indicated by the figures given by the engineer, measured from the grade stakes. At each grade stake, a firm support shall be erected; and on these supports a fine, stout cord shall be tightly stretched over the center line of the ditch and made parallel with the grade by careful measurements at each stake, using a carpenter's level. Supports shall be kept erected at at least three grade stakes, and the work checked each time by sighting over them. Intermediate supports shall be set and lined in by careful sighting wherever necessary, to support the cord every 50 feet. A suitable measuring stick shall be passed along the entire ditch, and the bottom in all parts made true to grade by measuring from the cord. The bottom must be dressed with the tile hoe, or, in the case of large tiles, with the shovel, so that a groove will be made to receive the tile, in which the tile will remain securely in place when laid.

4. *Laying the Tile.* The laying of the tile must begin at the lower end and proceed upstream. The tile must be laid as closely as practicable, and in lines free from irregular crooks, the pieces being turned about until the upper edge closes, unless there is sand or fine silt which is likely to run into the tile, in which case the lower edge must be laid close, and the upper side covered with clay or other suitable material. When in making turns, or by reason of irregular-shaped tile, a crack of one-fourth inch or more is necessarily left, it must be securely covered with broken pieces of tile. Junctions with branch lines must be carefully and securely made.

5. *Blinding the Tile.* After the tile have been laid and inspected by the employer or his representative, they must be covered with clay to a depth of six inches, unless, in the judgment of the employer or his representative, the tile are sufficiently firm so that complete filling of the ditch may be made directly upon the tile. In no case must the tile be covered with sand without other material being first used.

6. *Risk During Construction.* The ditch contractor must assume all risks from storms and caving-in of ditches; and when each drain is completed, it must be free from sand and mud before it will be received and paid for in full. In case it is found impracticable, by reason of bad weather or unlooked-for trouble in digging the ditch or properly laying the tile, to complete the work at the time specified in the contract, the time may be extended as may be mutually agreed upon by the employer and contractor. The contractor shall use all necessary precaution to secure his work from injury while he is constructing the drain.

7. *The Tile to be Used.* Tile will be delivered on the ground convenient for the use of the contractor. No tile shall be laid which are broken, or soft, or so badly out of shape that they cannot be well laid and make a good, satisfactory drain.

8. *Prosecution of the Work.* The work must be pushed as fast as will be consistent with economy and good workmanship, and must not be left by the contractor for the purpose of working upon other contracts, except by permission and consent of the employer. All survey stakes shall be preserved, and every means taken to do the work in a first-class manner.

9. *Subletting Work.* The contractor shall not sublet any part of the work in such a way that he will not remain personally responsible, nor shall any other party be recognized in the payment for work.

10. *Plant and Tools.* The contractor shall furnish all tools which are necessary to be used in digging the ditches, grading the bottom, and laying the tile. In case it is necessary to use curbing for the ditches, or outside material for covering the tile where sand or slush is encountered, the employer shall furnish the same upon the ground convenient for use.

11. *Payments for Work.* Every............weeks during the prosecution of the work, the contractor may claim and the employer shall pay 75% of the value of the work completed satisfactorily, the engineer being the arbiter in case of dispute as to the amount of work satisfactorily completed. The remaining 25% will be retained until the entire work is completed satisfactorily, as certified by the engineer after a final inspection, at which time the whole amount due shall be paid. Prior to any payment, the employer may require a correct statement of all claims incurred by the contractor for labor, materials, or damages on account of the work; and the employer may withhold payments until proof has been presented by the contractor of release of all liens against the employer on account of such claims.

12. *Duties of Engineer.* The engineer shall have authority to lay out and direct the work, and to inspect and supervise the same during construction and on completion, to see that it is properly done in accordance with the contract. His instructions should be fully carried out.

13. *Failure to Comply with Specifications.* In case the contractor shall fail to comply with the specifications, or refuse to correct faults in the work as soon as they are pointed out by the engineer or other person in charge, the employer may declare the contract void; and the contractor, upon receiving seventy-five per cent of the value of the completed drains at the price agreed upon, shall release the work and the employer may let it to other parties.

71. Benefits of Tile Drains. The advantages of tile drains may be enumerated as follows:

1. Tile drainage, by making the soil firm, enables earlier cultivation in the spring. Low ground drained can be cultivated earlier than high ground not drained.

2. Careful observations have shown that tile drainage makes the soil several degrees warmer in the spring. Scientific tests have

shown this increased warmth to be of the utmost importance in promoting the germination and growth of crops.

3. Tile drainage promotes pulverization of the soil, putting it in good condition to cultivate, and preventing baking and the formation of clods.

4. Tile drainage removes from the pores of the soil surplus and stagnant water, which would drown and destroy the roots of plants.

5. Tile drainage makes certain the proper "breathing" of the soil, or free circulation of air in its pores, which is essential to healthy plant growth.

6. Tile drainage establishes in the soil the proper conditions required for the satisfactory carrying on of the chemical processes necessary to prepare the plant food for its use by vegetation.

7. Tile drainage fits the soil for the vigorous life and action of the soil bacteria which are essential to preserve and increase its fertility and promote the growth of crops.

8. Tile drainage increases the depth of soil which can be reached by the roots of plants and drawn upon for plant food.

9. Because in them the roots of plants can penetrate deeper, where they are protected from heat and drouth and can reach the deep-seated moisture, tile-drained soils stand drouth better than undrained soils.

10. By putting the top 3-feet or 4-feet layer of soil into a porous condition, tile drainage enables soils to absorb rain water instead of discharging it over the surface, and so helps to prevent surface wash and consequent loss of fertility.

11. By causing this porous condition, tile drainage makes the upper 3 or 4 feet of soil into an enormous reservoir to catch the rain water and discharge it only slowly into the streams. Thus tile drainage prevents floods instead of causing them.

12. Tile drainage does away with irregular shaped fields, cut up by sloughs and ditches, and so cheapens cultivation.

Benefits of Large Ditches. Tile drainage is always preferred to open-ditch drainage if the drain is not too large. The advantages of large ditches may be enumerated as follows:

1. By furnishing channels to remove storm water, they prevent, if of ample size, the inundation of low-lying lands by floods and surface water.

2. They have a minor value for draining off the ground water from a narrow strip of land each side.

3. One of their main values is in furnishing outlets for tile drains, and in many places tile drainage is impracticable till outlet drainage ditches have been built.

72. Method of Computing Sizes of Tile Drains. The drained soil above the level of tile drains contains a large percentage of air-space in the pores between the soil particles; and this layer of porous soil acts like a great sponge several feet thick to absorb the rain as it falls. Hence the water reaches the tiles very slowly. It has been found that under average conditions tiles will not be called upon to carry more than ¼-inch depth of water in 24 hours. This equals 6,800 gallons per acre per day, or 4,352,000 gallons per square mile per day. The sizes of tile drains for average conditions may readily be taken from Table IX.

TABLE IX

Number of Acres Drained by Tiles Removing ¼-Inch Depth of Water in 24 Hours

Grades		Diameters of Tile Drains										
Per cent	Inches per rod	3 in.	4 in.	6 in	8 in.	10 in.	12 in.	15 in.	18 in.	20 in	22 in.	24 in
0.03	1/16					37	59	109	159	205	254	319
0.05	3/32		5	13	28	49	75	131	219	261	332	411
0.10	3/16	4	7	19	40	69	109	186	289	373	471	582
0.15	9/32	4	9	24	49	85	132	232	355	458	577	713
0.25	3/8	5	10	28	56	97	153	264	410	529	667	823
0.30	9/16	6	12	33	69	119	188	322	502	618	808	1,008
0.40	13/16	7	14	39	79	138	216	371	580	718	942	1,165
0.50	1	8	16	44	89	151	246	416	648	838	1,050	1,300
0.60	1 3/16	9	17	48	97	169	266	457	710	911	1,154	1,422
0.70	1 3/8	10	19	50	105	182	287	488	768	988	1,242	1,549
0.80	1 9/16	10	20	55	114	195	307	526	822	1,059	1,332	1,615
0.90	1 3/4	10	21	59	119	207	326	558	872	1,123	1,414	1,747
1.00	2	11	22	62	126	218	343	589	917	1,176	1,495	1,838
1.50	3	13	28	75	153	267	419	722	1,123	1,450	1,824	2,256
2.00	4	15	31	88	178	309	485	832	1,297	1,676	2,110	2,594
3.00	5 15/16	19	39	107	216	377	593	1,020	1,589	1,957	2,592	
4.00	7 13/16	22	45	123	253	437	683	1,176				
5.00	9 7/8	25	50	138	280	486	765					
7.50	14 7/8	30	61	169	344							
10.00	19 13/16	35	71	195								

Table IX is computed from the form of Poncelet's formula recommended for use with tile drains by C. G. Elliott, drainage expert to the U. S. Agricultural Department, Washington, D. C., who recommends the above sizes to drain

ground water only. If surface water is also to be removed, as in the case of ponds without other outlets, the tiles will drain safely only one-half to one-third the number of acres given in the table.

When part of the land in the watershed is rolling, not requiring tiling, count only one-fifth to one-third of such rolling land, in addition to all of the low, flat land, in getting the size of tiles to remove ground water only.

Example 41. What size of tile laid to a 0.1 per cent grade will carry the under-drainage of 160 acres of flat land?

Answer. 15 inches.

Example 42. What size of tile to a 0.2 per cent grade will carry the under drainage of 240 acres, two-thirds rolling?

Answer. 80 acres flat land, *plus* one-third of 160 acres rolling, gives 133⅓ acres, requiring a 12-inch tile.

Example 43. What size of tile laid to 0.3 per cent grade will be required to remove both ground and surface water from a pond whose watershed includes 40 acres?

Answer. 10-inch. (NOTE.—Double or triple the area for both ground and surface water.)

73. Method of Computing Sizes of Drainage Ditches. Since drainage ditches must carry surface water as well as ground water, their capacities must be larger than those of tile drains for the same number of acres drained. It has been found by experience that they must carry from ¾-inch depth for small drainage areas, to ¼-inch depth for large drainage areas per day. Their size can be taken from Table X.

Example 44. What width of ditch, having a fall of 5 feet per mile, and a depth of water of 3 feet, will be required to drain an area of 5 square miles (3,200 acres)?

Answer. About 12 feet.

Example 45. What size ditch having a fall of 3 ft. per mile, and 9 ft. depth of water, will drain an area of three townships (69,120 acres)?

Answer. About 22 feet.

74. Method of Computing Sizes of Subdrains for Sewers. Sewer subdrains act like tile land drains to remove the ground water from the soil. Being deeper, they will drain wider strips of land—say averaging 16 rods wide, instead of 8 rods, for ordinary land drains in average soil; but also, owing to the greater depth, the water will reach the tiles more slowly, and this may offset the greater width drained. We may assume roughly that each subdrain may be called upon to remove ⅛-inch depth of water per day from a strip 16 rods wide, *which is the same thing as ¼-inch depth per day from a strip of land 8 rods wide.*

TABLE X

Number of Acres Drained by Open Ditches

Grade		*Depth of Water, 3 feet. Depth of Ditch, at least 4 feet.* Average Width of Water								*Depth of Water, 5 feet. Depth of Ditch, at least 6.5 feet.* Average Width of Water						
Per cent	Feet per Mile	4 feet	6 feet	8 feet	10 feet	15 feet	20 feet	30 feet	50 feet	6 feet	8 feet	10 feet	15 feet	20 feet	30 feet	50 feet
0.02	1.0			725	970	1,570	2,240	5,300	18,400	980	1,470	1,900	5,000	7,150	23,800	43,800
0.04	2.1	400	690	1,000	1,360	2,250	4,700	7,470	26,100	1,390	2,090	2,800	7,200	20,400	33,500	62,500
0.06	3.2	492	850	1,260	1,690	2,770	5,770	18,400	31,900	1,710	2,560	5,100	17,600	24,700	40,800	75,500
0.08	4.2	572	980	1,460	1,950	4,820	6,670	21,400	37,400	1,980	2,980	6,100	20,400	30,000	48,800	88,000
0.10	5.3	636	1,100	1,630	2,180	5,360	7,440	23,700	41,400	2,220	5,010	7,600	23,400	33,400	54,500	98,000
0.15	7.8	791	1,330	2,010	2,670	6,600	19,000	30,200	52,100	2,720	6,300	17,100	28,700	40,500	66,700	120,000
0.20	10.6	905	1,560	2,310	4,720	7,870	21,800	35,000	60,300	4,820	7,300	19,500	33,000	47,000	77,000	139,000
0.25	13.2	1,020	1,740	2,660	5,300	17,500	24,600	39,000	67,700	5,370	16,300	21,900	37,500	53,000	86,000	155,000
0.30	15.8	1,100	1,970	2,900	5,850	19,400	26,800	42,700	74,000	5,900	17,900	23,900	40,700	57,000	94,000	170,000
0.40	21.1	1,300	2,290	5,050	6,740	22,200	30,800	49,400	85,700	6,830	20,600	27,700	47,000	67,000		
0.50	26.4	1,475	2,550	5,620	7,500	24,800	34,800	55,300	95,200	7,600	23,000	31,000				
0.60	31.7	1,600	2,790	6,230	16,500	27,200	37,700	60,400		16,700	25,200	33,900				
0.70	37.0	1,720	3,010	6,650	17,800	29,400	41,200			18,100	27,300					
0.80	42.2	1,850	4,850	7,170	19,100					19,000						
0.90	47.5	1,955	5,140	7,550	20,100					20,500						
1.00	52.8		5,400													

TABLE X—(Concluded)

Number of Acres Drained by Open Ditches

Depth of Water, 7 feet.					*Depth of Ditch, at least 9 feet.*		*Depth of Water, 9 feet.*		*Depth of Ditch, at least 11.5 feet.*		
Grade		Average Width of Water						Average Width of Water			
Per cent	Feet per mile	8 feet	10 feet	15 feet	20 feet	30 feet	50 feet	10 feet	15 feet	20 feet	30 feet	50 feet
0.02	1.0	2,300	4,700	16,600	28,000	48,000	88,500	6,550	27,800	40,800	69,500	127,000
0.04	2.1	4,850	6,740	23,400	35,400	58,000	106,000	18,500	34,400	50,000	83,500	157,000
0.06	3.2	5,920	17,000	29,600	43,400	72,000	129,000	22,600	41,600	61,000	103,000	193,000
0.08	4.2	6,940	19,100	34,200	50,000	83,000	150,000	26,300	48,300	71,000	120,000	221,000
0.10	5.3	7,720	21,800	38,400	56,000	92,600	167,000	30,400	54,000	79,100	132,000	244,000
0.15	7.8	19,400	27,000	47,200	68,500	112,000	202,000	37,300	66,100	96,200	162,000	298,000
0.20	10.6	22,400	31,300	54,200	78,700	130,000	235,000	42,900	76,200	104,000		
0.25	13.2	25,000	34,800	60,500	88,000	146,000		48,000	85,300	125,000		
0.30	15.8	27,400	38,200	66,200	96,500			52,500	93,200			
0.40	21.1	31,700	44,100					60,800				
0.50	26.4	35,400										

Table X, for open ditches, is calculated by the well-known standard Kutter's formula, using a "coefficient of roughness" equal to 0.030. This coefficient of roughness is the value recommended by Kutter for channels in moderately good condition, having stones and weeds occasionally, and agrees with actual gaugings of drainage channels made at the Iowa State College. For ditches in first-class condition, the number of acres given may be increased about 25 per cent. The table has been calculated for ditches having sides with slopes of one foot horizontal to one foot vertical but is approximately correct for other slopes.

The capacity of the ditches has been made as recommended by C. G. Elliott, U. S. Agricultural Department drainage expert, as follows, the ditches to run not more than $\frac{8}{10}$ full for the capacities mentioned:

Above the upper heavy line, $\frac{3}{4}$-inch depth of water per 24 hours
Between the two heavy lines, $\frac{1}{2}$-inch depth of water per 24 hours.
Below the lower heavy line, $\frac{1}{4}$-inch depth of water per 24 hours.

Local conditions may vary the size needed, and it is necessary to consult a drainage engineer in each case.

Hence the sizes required for sewer sub-drains may be taken from Table IX, calculating the number of acres drained by multiplying the total lengths of tributary drain tile, in feet, by 132 feet (= 8 rods), and dividing the product by 43,560 sq. ft.

The above method will give a capacity approximating 110,000 gallons per day per mile of tributary subdrains. As sewers are ordinarily distributed, it will give a capacity approximating 1,500,000 gallons per day per square mile of territory served by the sewers.

Example 46. Calculate the size of subdrains laid to a 0.25 per cent grade, required to serve as outlet for 30,000 linear feet of tributary subdrains.

Solution: $\dfrac{30{,}000 \times 132}{43{,}560} = 91$ acres = equivalent area drained for $\frac{1}{4}$-inch depth.

In Table IX, opposite the 0.25 per cent grade, we find that a 10-inch tile would be required.

Answer. 10-inch tile subdrain.

75. Cost of Tile Land Drains and Drainage Ditches. The cost of tile-drain construction in central Iowa in 1904, can be approximated from Table XI. Local prices should be determined before using the table for close estimates of work done elsewhere.

TABLE XI
Cost of Tile Drains

Size of Tile	Price per 1,000 Feet	Weight per Foot	Cost of Hauling 1,000 Feet 5 Miles	Cost of Digging and Laying, per Rod: 3 feet deep or less	Add per foot for additional depth over 3 feet: 3–6 ft.	Add per foot for additional depth over 3 feet: over 6 ft.	Refilling, per Rod
3 in.	$ 16.00	5	$ 3.12	$ 0.35	$ 0.15	$ 0.30	2c.-5c.
4 in.	22.00	8	5.00	0.35	0.15	0.30	2c.-5c
5 in.	30.00	10	6 25	0.35	0.15	0.30	2c.-5c.
6 in.	40.00	12	7.50	0.35	0.15	0.30	2c.-5c.
7 in.	50.00	15	9.37	0.35	0.20	0.35	2c.-5c.
8 in.	60.00	20	12.50	0.40	0.20	0.35	2c.-5c.
10 in.	95.00	30	18.75	0.45	0.20	0.35	2c.-5c.
12 in.	120.00	40	25.00	0.50	0.20	0.35	2c.-5c.
15 in.	250.00	50	31.25				
18 in.	400.00	80	50.00				
20 in.	600.00	100	62.50				
24 in.	800.00	125	78.12				

The cost of hauling given in Table XI is on the basis of \$1.25 per ton, or \$2.50 per day for a man and team, making two trips.

The prices for digging and laying given above include board furnished by the ditcher. If the farmer furnishes board, deduct about 20 per cent. The prices for digging and laying are for average ground, and should be increased for quicksand or very wet soils.

N. B. To all estimates it is wise to add 5 per cent to 10 per cent for contingencies and engineering.

Example 47. What will be the cost of 2,000 feet of 6-in. tile drain, 2½ miles from the tile yard, of which 1,000 feet is 4 feet deep, 500 feet 5 feet deep, and 500 feet 6 feet deep, in average soil?

Answer:

2,000 ft. of 6 in. tile @ \$40.00	\$80
Hauling 2,000 ft. 2½ miles, @ \$3.75	7½
Digging and laying 60.6 rods 4 ft. deep, @ 50c	30½
" " " 30.3 rods 5 ft. deep, @ 65c	19½
" " " 30.3 rods 6 ft. deep, @ 80c	24
Refilling 121.2 rods (by team), @ 2c	2½
	\$164
Add 10 per cent for engineering, etc.	16
Estimated cost	\$180

Cost of Open Drainage Ditches. The cost of open drainage ditches is estimated by the cubic yard.

To calculate the number of cubic yards per foot of length of ditch, multiply the average width by the average depth, and divide by 27. Thus a 7-ft. by 12-ft. ditch contains $\frac{7 \times 12}{27} = 3\frac{1}{9}$ cubic yds. per foot length.

The cost per cubic yard in Iowa varies from 7c. to 18c., depending on the size of the job, the character of the soil, and other local conditions, including the certainty of the contractor getting his money promptly. The larger the work, the less is the cost per cubic yard.

HOUSE SEWERAGE

76. Definitions and General Description. A *house sewer* is a small branch sewer which connects the house with the street sewer. In Fig. 6 a general view of a house sewer is given.

A *soil pipe* is the main drainage pipe of the system of house plumbing, into which the different fixtures discharge. See Fig. 35.

A *trap* is a bend or depression in a pipe or drain, which remains constantly full of liquid, thus shutting off air-connection between the portions of the pipe or drain on opposite sides of the trap. See Fig. 35.

A general idea of an entire system of house sewerage can be obtained from Figs. 6 and 35, which see.

The house sewer and outlet for the cellar and foundation drains, extend from the street sewer to the house as shown in Fig. 6.

The iron soil pipe should begin a few feet outside the house, and extend full size through the roof, the separate fixtures discharging into the soil pipe, each protected by a trap, and all traps being vented, as shown in Fig. 35. The dotted lines in Fig. 35 show alternative plans sometimes adopted for house sewerage.

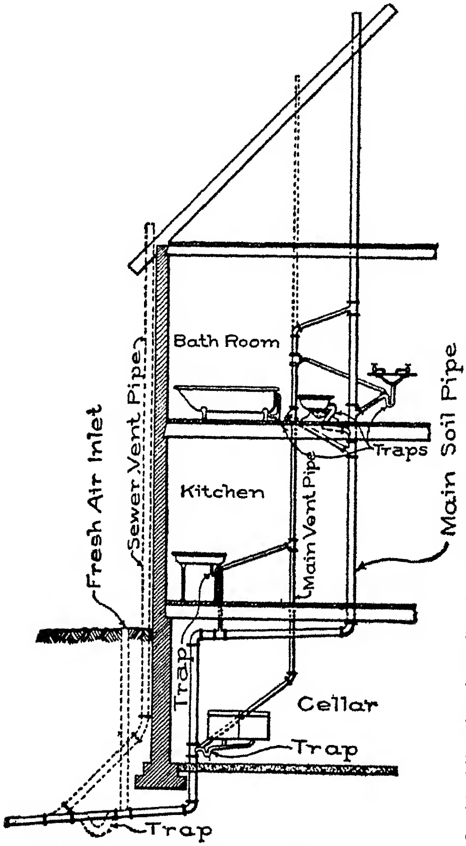


Fig. 35. Diagram of House Sewerage System.

77. House Sewers. House sewers (see Fig. 6) are usually made of vitrified sewer pipe the same as street sewers, and should be constructed with fully as much care. The joints should have gaskets of hemp or oakum, and be carefully cemented, the same as street sewers. (See Art. 33.)

Each piece of pipe should be laid to the exact grade by measuring from a grade string, the same as for street sewers (see Art. 98). The grade should usually be not less than 2 per cent. The house sewer should, if possible, be perfectly straight, both in alignment and in grade, from the house to the house connection at the sewer.

Inspection pipes should be placed just inside the lot line, as indicated in Fig. 6.

House sewers should usually be 4-inch circular pipe. If too large, they are more difficult to keep flushed clean, and they may carry to the street sewer things large enough to cause stoppages, improperly put into the house fixtures. Sometimes 5-inch or 6-inch house sewers are used.

78. General Principles of House Plumbing. The following general principles should be carefully observed in the installation of all house plumbing:

1. The iron pipe should begin a few feet outside the house, as vitrified pipe does not have tight joints and is liable to be broken, where it passes through the foundation wall, by uneven settlement.

2. No pipes carrying sewage should be allowed to be buried under the basement floor, unless placed in masonry-lined trenches with removable covers.

3. All pipes of the plumbing system should be iron or lead, with absolutely tight joints of lead, or screwed, or soldered.

4. In general, no pipes should be built into partitions or walls, where they cannot be gotten at, unless removable panels are placed over them.

5. All fixtures should be completely exposed to view, and should not be enclosed in woodwork. Sinks and washbowls, for example, should be supported on brackets or legs, with clear, open spaces under them.

6. All fixtures should be of durable, smooth, and non-absorbent material, such as porcelain or enameled iron. The least possible woodwork should be used.

7. All fixtures should be located in well-lighted and well-ventilated places.

8. Each fixture must be protected by a good trap. There must be no openings from the plumbing system into the interior of the house not thoroughly protected by traps sure to stav full of liquid.

9. Thorough ventilation of all pipes must be provided for.

10. All pipes must be laid to good grades, without sags, so as to drain completely and quickly.

11. The cellar and foundation drains should be connected with a sewer subdrain, if possible, and not with a sewer, owing to the danger of the water in the traps evaporating in dry weather when no water runs in the drains. If absolutely necessary to connect to the sewer, excessively deep traps should be used, to lessen the danger of evaporation.

79. Soil Pipes. The iron soil pipe begins, as already stated, a few feet outside the foundation wall. At this point a *disconnecting trap* is sometimes placed, as shown by the dotted lines in Fig. 35, in which case a *fresh-air inlet* must be placed on the house side of the

trap, as also shown by dotted lines in Fig. 35, to permit complete ventilation of the soil pipe.

The soil pipe should extend full-sized and without any obstruction, a few feet above the roof. It should everywhere be readily accessible, and will naturally be placed in the location most convenient for attaching the fixtures.

The soil pipe is usually 4 inches in diameter, made of cast iron, with air-tight, leaded and calked joints.

80. Traps. The best traps are simply *smooth bends* in the plumbing pipes, giving depressions which stand full of liquid. If the curves are not smooth, or if there are sudden changes in size, the danger of stoppage is increased. The depth from the highest level of the water in the trap to the top of the liquid in the lowest portion, is called the *seal* of the trap. Traps are necessary evils in plumbing systems, as they tend to cause stoppages.

The seals of traps may be forced by any compression or rarefaction of air in the plumbing pipes, such as may be caused by *plugs* of sewage from other fixtures descending the pipes, unless a *vent pipe* is extended from the *crown* or highest point of each trap on the side next to the soil pipe, as shown in Fig. 35.

Traps should be located as closely as possible to the fixtures they are to protect.

81. Ventilation. The vent pipes from the traps mentioned in Art. 80, above, and shown in Fig. 35, serve also to secure ventilation of branch pipes. They should unite in a *main vent pipe*, 2 inches in diameter, as shown in Fig. 35, and this may turn into the soil pipe above the highest fixture, or may extend independently above the roof, as shown by the dotted lines in Fig. 35.

The extension of the main soil pipe unobstructed through the roof, with admission of air from the sewer (or through the fresh-air inlet if a disconnecting trap is used), together with the trap vent pipes and the main vent pipe, as shown in Fig. 35, insure ventilation of all parts of the plumbing system.

COST OF SEWERS, AND METHODS OF PAYING FOR THEM

82. Preliminary Estimates of Cost of Sewers. One of the first things which the sewerage engineer will be asked about sewers for

which he has made plans, is what will be their cost. He must be able to answer this question readily, and with close approximation to the actual cost.

Many factors affect the cost of sewers, some of which cannot be exactly foretold. Among the things which can be closely ascertained in advance, are the sizes, lengths, and depths of the sewer, and the amounts of the various kinds of materials required. Among the things which cannot be exactly foretold, are the nature of the soil, the amount of ground water to be encountered, the weather conditions, and the labor conditions.

The competent engineer will thoroughly study all conditions which may affect the cost, before preparing his estimates, and even then will allow a liberal percentage for contingencies.

The engineer should have borings made to determine the character of the soil and the level of ground water, and should learn all he can of previous experience in the town with ditches and other excavations. Even then the actual soil often proves very different from what was anticipated.

After making the preliminary study and plans, the engineer tabulates the sewers by lengths, depths, sizes, and character, together with the manholes, lampholes, flush-tanks, and other items of the system. He then assigns a unit price to each item, after careful study of all conditions, and calculates the total cost.

The data of cost which follow are for average conditions only, and only for the localities named. They will need to be modified by the engineer to meet different conditions.

83. Cost of Pipe Sewers. In estimates of the cost of pipe sewers, the work is usually divided into the following items:

(1) *Trenching and Refilling.* This includes excavating the trench for the sewer, refilling it, and compacting the material after the sewer pipe is laid. Trenching and refilling are usually itemized according to depth, thus:

Trenching and Refilling under 6 feet depth
" " " " 6 to 8 feet depth
" " " " 8 to 10 feet depth
Etc., etc.

The cost of trenching and refilling will vary somewhat also with the diameter of the sewer; but this is often not separately itemized.

For estimates and bids, the lengths in linear feet of each depth of sewer are taken from the profiles, and listed in the tabulation.

(2) *Furnishing Sewer Pipe and Specials.* The pipe are usually specified to be delivered on board cars at the town where they are to be used. The amounts are usually itemized according to the diameters, thus:

Furnishing	sewer	pipe	8	inches	diameter
"	"	"	10	"	"
"	"	"	12	"	"
etc., etc.					

Specials are sometimes itemized separately, and sometimes included in the prices for furnishing pipe, the average distance apart being specified.

For estimates and bids, the total lengths of each size of pipe are ascertained and listed in the tabulation.

(3) *Hauling and Laying Sewer Pipe and Specials.* This includes taking the sewer pipe from the cars, hauling them to the sewer, furnishing cement, sand, and hemp or oakum, and laying the pipe according to the specifications. Some labor in excavating bell holes and a few inches at the bottom of the ditch shaped to fit closely the under side of the pipe, is also included. Hauling and laying are usually itemized according to the diameters of the pipe, thus:

Hauling	and	Laying	sewer	pipe	and	specials,	8	inches	diameter
"	"	"	"	"	"	"	10	"	"
"	"	"	"	"	"	"	12	"	"
Etc., etc.									

The lengths of each size are listed for estimates and bids, the same as sewer pipe.

In Fig. 36 is given a diagram for estimating the cost of pipe sewers and subdrains in the Middle West. It may be used elsewhere by noting local conditions and their variation from the conditions assumed, as follows:

(*a*) If the sewers are to be paid for promptly as the work progresses, in cash instead of in assessment certificates, deduct about 10 per cent.

(*b*) Get actual prices on sewer pipe delivered, and add about 8 per cent for additional cost of specials in the average residence district, and 16 per cent in the average business district.

(c) Ascertain the character of the soil, and the likelihood of encountering ground water. If the conditions are very favorable, the cost of trenching, refilling, and pipe laying may be materially decreased, even sometimes to 50 per cent of the figures shown in the diagram; while on the other hand, for very unfavorable conditions, the cost shown for these items will have to be increased, sometimes even to 150 per cent.

Example 48. Estimate the cost of a pipe sewer consisting of 1,200 ft. of 18-inch pipe averaging 16 feet deep, and 2,700 feet of 15-inch pipe averaging 12 ft. deep, under average conditions, together with a 6-inch subdrain.

Solution:

1,200 × 2.35 (from diagram) =	$3,020	for 18-inch	sewer
2,700 × 1.60 (" ") =	4,320	" 15 "	"
3,900 × 0.15 (" ") =	585	" 6 "	subdrain
Answer. Total estimated cost =	$7,925		

84. Cost of Brick Sewers. The cost of a brick sewer may be estimated by determining separately the cost of the excavation and refilling and that of the brickwork. The number of cubic yards of each of these items is computed for 1 linear foot length of sewer; and the cost per linear foot is estimated by multiplying the results so obtained by estimated costs per cubic yard of excavation and brickwork respectively.

(1) *To calculate the number of cubic yards of excavation per linear foot length of sewer, multiply the average depth of sewer trench by the average width, and divide by 27.*

The *average depth* for a circular bottom will approximate the *average depth from the surface to the invert,* while the *average width* will be *at least as great as the internal diameter plus twice the thickness of the brickwork.*

Thus, for a 2-ring (9 inches of brickwork) circular sewer 6 feet in diameter, with grade line 12 ft. deep, the number of cubic yards excavation per linear foot of sewer is:

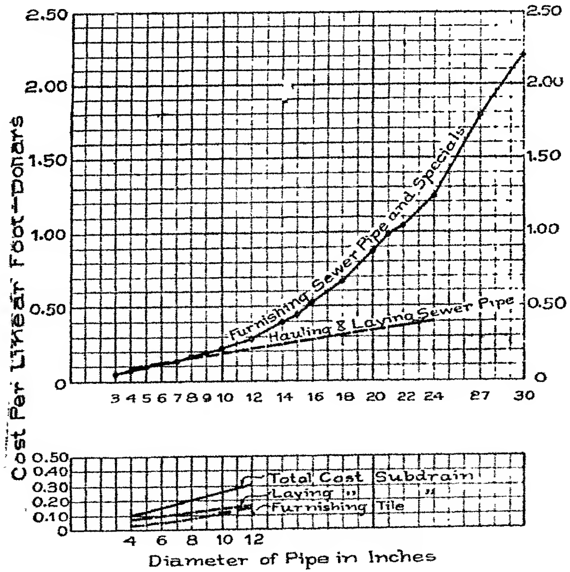
$$\frac{12 \times (6 + 1\frac{1}{2})}{27} = \frac{90}{27} = 3\tfrac{1}{3} \text{ cu. yds. per linear ft.}$$

The cost of sewer excavation and refilling varies usually from $0.20 *per cu. yd. to* $1.20 *per cu. yd., averaging perhaps* $0.50 *to* $0.75 *per cu. yd.*

DIAGRAMS FOR ESTIMATING COST OF PIPE SEWERS

Prepared from data collected for average conditions in the Middle West, 1906; ditches braced and partially sheathed; amount of water, moderate; common labor, $1.75 a day; payments to contractor, partly in assessment certificates.

For especially favorable or unfavorable conditions, the cost may vary 50 per cent either way, except for furnishing.



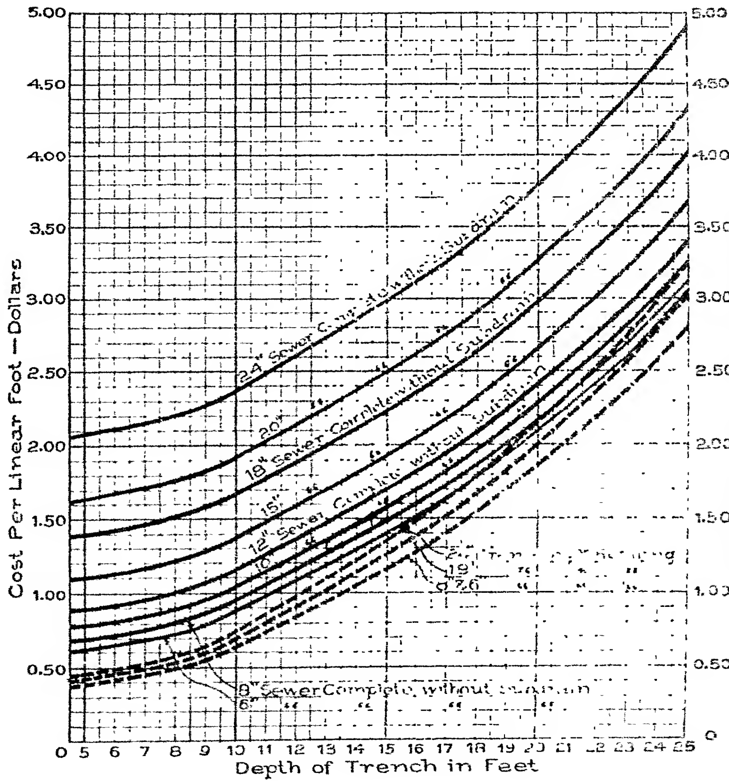


Fig. 36. Cost of Pipe Sewers.

Thus, for average conditions, fairly favorable, the cost of excavation for the 6-foot sewer, 12 feet deep, referred to above, would be $3\frac{1}{3} \times .60 = \$2.00$ per linear foot.

The favorable conditions for low cost per cubic yard, are, large sewers; neither great shallowness nor excessive depth; little water; soil firm enough not to require much bracing, yet not hard enough to require to be picked; and the use of excavating machinery. The opposites of these conditions give the unfavorable conditions.

(2) *The number of cubic yards of brickwork per linear foot* of brick sewers, may be taken from Tables XII and XIII, which are taken mainly from Gillette's *Handbook of Cost Data.*

TABLE XII

Cubic Yards per Linear Foot of Brick Masonry in Circular Sewers

Diameter	One Ring	Two Rings	Three Rings
2 ft. 6 in.	0.125	0 283	
3 " 0 "	0.147	0.327	
3 " 6 "	0 169	0.371	
4 " 0"	0.191	0.415	
4 " 6 "	0.213	0 418	
5 " 0 "	0.234	0 502	0 802
5 " 6 "	0.256	0 544	0 867
6 " 0 "	0.278	0 589	0 933
6 " 6 "		0 633	0.998
7 " 0 "		0 677	0 063
7 " 6 "		0.720	0 128
8 " 0 "		0.764	1.194
8 " 6 "		0.807	1.260
9 " 0 "		0.851	1.325
9 " 6 "		0.895	1.390
10 " 0 "		0.938	1.456

TABLE XIII

Cubic Yards per Linear Foot of Brick Masonry in Egg-Shaped Sewers

Dimensions	One Ring	Two Rings	Three Rings
ft. in. ft. in.			
2–0 by 3–6	0.128	0.286	
2–6 " 3–9	0.154	0.341	
3–0 " 4–6	0.182	0.396	
3–6 " 5–3		0.451	0.725
4–0 " 6–0		0.506	0.808
4–6 " 6–9		0.561	0.891
5–0 " 7–6		0.617	0.974
5–6 " 8–3		0.673	1.056
6–0 " 9–0		0.729	1.140
6–6 " 9–9		0.785	1.223

The cost of brick masonry in sewers usually varies from \$8.00 *to* \$14.00 *per cubic yard, averaging perhaps* \$9.50 *to* \$12.00.

Thus, under average conditions, the cost, per linear foot, of the brick masonry of the two-ring, 6-foot circular brick sewer mentioned above, would be about 0.589 cu. yds. (from Table XII) × \$10.50 per cu. yd. = \$6.17 per foot. It will depend upon the grade of brick used, their cost per 1,000, the cost and proportions of cement and sand in the mortar, the wages of brick masons, the size and depth of the ditch, etc.

Example 49. Estimate the cost, under fairly favorable conditions, as to excavation and brickwork, of a 10-foot, 3-ring, circular brick sewer 1,875 ft. long, averaging 10 ft. deep.

Solution:

$$\text{Cu. yds. excavation per foot} = \text{about } \frac{10 \times 13}{27} = 5$$

(allowing 13 ft. width of trench, to provide a little extra room for bracing).

Since the conditions are fair, assume \$0.60 per cu. yd. as cost of excavation and refilling.

The brickwork = 1.456 cu. yds. per linear foot (Table XII); and since the conditions are fair, we shall assume a cost of \$9.50 per cu. yd.

Then the estimate will be as follows:

Excavation and Refilling,	5 × \$0.60 =	\$ 3.00 per lin. ft.
Brickwork	1.456 × 9.50 =	13.83 " " "
Total		\$16.83 " " "

1,875 × 16.83 = \$31,556 for total cost, to which, however, it may be wise to add, say, 5 to 10 per cent for contingencies unforeseen.

Answer. About \$33,500.

85. Cost of Concrete Sewers. The cost of concrete sewers may be estimated by a method precisely similar to that described in Art. 84, above, for brick sewers—namely:

(1) *Compute the cubic yards of excavation per linear foot of sewer* $\left(= \frac{\textit{average depth} \times \textit{average width})}{27}\right)$, *and multiply by the estimated cost per cubic yard, which will be from* \$0.20 *to* \$1.20, *usually* \$0.50 *to* \$0.75.

(2) *Compute the number of cubic yards of concrete per linear foot of sewer*

$$\left(= \frac{\textit{total area of concrete in square feet in a cross-section of the sewer}}{27} \right)$$

and multiply by the estimated cost of the concrete per cubic yard, which will be from \$6.50 *to* \$12.00, *usually from* \$7.50 *to* \$9.50.

(3) *In the case of reinforced concrete sewers, compute the number of pounds of steel reinforcing per linear foot of sewer, and multiply by* \$0.04 *to* \$0.05 *per lb.*

The details of designs for concrete and reinforced concrete sewers vary so much that no tables can be given, as for brick sewers, showing the cubic yards of concrete per linear foot of sewer.

The cost of the concrete will depend upon the costs of cement, sand, and broken stone or gravel, and on their proportions; on the size and depth of the trench and its freedom from water; on the cost of labor, etc.

86. Cost of Manholes, Combined Manholes and Flush-Tanks, Flush-Tanks, Lampholes, and Deep-Cut House Connections. Under these headings the following data of cost will be found valuable:

Manholes. Under average conditions, the cost of brick manholes of the design shown in Fig. 9, will be *about* \$40 *for* 8 *ft. depth of sewer.* For greater depths, *add about* \$3 *per foot of additional depth.*

Combined Manholes and Flush-Tanks. Under average conditions, the cost of these may be estimated at \$80, *plus* \$4 *per foot of additional depth of sewer over* 8 *ft.* This is for about 500 gallons' capacity of the flush-tank part.

Flush-tanks of 500 gallons' capacity, under average conditions, may be estimated to cost *about* \$60 *each.*

Lampholes, such as shown in Fig. 10, may be estimated at *about* \$10, *plus* \$0.35 *per foot of additional depth over* 8 *feet.*

Deep-cut house connections (see Fig. 8) may be estimated at \$2.00 to \$3.00 *each,* according to the depth of the sewer.

87. Engineering and Contingencies. In estimates of the cost of a sewer system, it is necessary to allow for unforeseen contingencies and for the cost of the engineering work. From 5 per cent to 20 per cent is usually added to the estimated cost on these accounts, depend-

ing upon the certainty or uncertainty of the knowledge of all the conditions.

EXAMPLE FOR PRACTICE

88. *Example* 50. Estimate the cost of the sewer system shown below, the conditions being assumed to be average. (NOTE: See Articles 84 to 87, inclusive.)

PRELIMINARY ESTIMATE OF COST OF SEWER SYSTEM FOR

____

Item	Approx Quantity	Cost	
		Unit	Total
4-ft. brick sewer, 2 rings, 8 ft. average depth	850 ft.		
3-ft. " " 2 " 10 " " "	625 "		
24-in. pipe sewer, 9 ft. average depth	3,780 "		
18 " " " 11 " " "	1,740 "		
12 " " " 14 " " "	2,640 "		
8 " " " 10½ " " "	46,800 "		
Manholes 12 " " "	68		
Comb. M.H.& F.T.10 " " "	18		
Lampholes 11 " " "	38		
Total of above			
Engineering and Contingencies, 10 per cent of above,			
Total estimate of cost			*

* Answer. About $82,500.

89. **Methods of Paying for Sewers.** This is another question which comes up early in determining whether a city can or will build or extend a sewer system.

Three methods are in common use in paying for sewers, as follows:

(1) *The City as a whole may pay the entire cost.* When this plan is followed, all or part of the money may be raised by selling bonds, or all or any part may be raised at once by taxation.

In some States, cities are given a right to levy a *sewer tax* of a certain rate for a certain number of years in advance, and to anticipate the proceeds of this tax by issuing *sewer warrants*.

Often, when it comes to the construction of sewers, the City will be found to have already issued bonds to the highest legal amount, to build waterworks, an electric light plant, etc., so that no money for sewers can be raised from bonds.

(2) *The entire cost of the sewers may be assessed against the property abutting upon or adjacent to the sewer.* Here the legal principle is that the assessment must be in proportion to the benefit received. Property abutting directly upon the sewer receives the greatest benefit, and must be assessed for most of the cost. Sometimes the benefit will be in proportion to the number of feet frontage of the lots abutting on the sewer; and sometimes the benefit per unit lot is considered to be the same in all parts of the city, a large unit size of lot being adopted in the residence part of the city, and a much smaller size in the business section, with often an intermediate size between these two.

The "assessment" is levied upon the completion of the sewer, when the entire cost can be ascertained. Due notice to all property owners assessed must be given, so that they can present objections if they desire. Usually all property owners who desire are allowed to spread the payment of their assessments in equal installments over a considerable period of years, in which case *assessment certificates* are issued to cover the payments. The contractor is often required to take these certificates in payment for the sewer.

(3) *The cost of the sewers may be divided between the City and the property directly abutting upon or adjacent to the sewer.* This seems the fairest way; since, in the first place, the entire city receives benefit from improved sanitation, attractiveness to investors, etc., from a sewer constructed anywhere within its limits; and since, in the second place, any system of sewers for a city should be planned to give outlets of proper size to all parts of the district, which enlarges and deepens the sewers on many streets. On the other hand, the property along the sewer is benefited much more than the rest of the city, and should accordingly pay a much larger proportion of the cost.

The City Council usually has the right to decide what percentage of the cost is to be paid by the City and what by the property along the sewers.

PREPARATION OF PLANS AND SPECIFICATIONS FOR SEWERAGE SYSTEMS

90. Sewer Reconnaissance. When a sanitary engineer is called upon to prepare plans and specifications for a sewerage system, the first thing which he should do is to make a *reconnaissance* or

general study of the entire city and its surroundings, with special reference to its sewerage conditions.

He visits the city and obtains copies of the best *maps* procurable. If these maps do not show the contours or elevations of the surface at different points, he obtains the best procurable information as to such elevations, and enters it upon the maps. Often the elevations of *street grades* will prove sufficient, if better and more detailed information is lacking. If *street profiles* are available, they will of course be of great value.

With maps thus prepared for the purpose, *he rides or walks over all parts of the city,* making himself thoroughly familiar with its *topography* and other features. Some of the information thus obtained may be entered upon the maps. He will note the *present density of population in different sections, and the prospects for future growth.* The presence or absence of *manufacturing industries,* and the future prospects in this line, are of importance. Statistics of the *past growth* of the city will be obtained. Full information regarding the character of the *water supply* and the amount and fluctuations of the *water consumption,* and the distribution of the *water mains* throughout the city, will be of great value. The local *labor conditions,* and the probable *local cost of cement, sand, brick, sewer pipe,* and other needed materials, must be ascertained. All possible information should be secured regarding the *ground water* and the *character of the soil* in different sections of the city. Information about old excavations and about wells can usually be secured, and will give much light on these points.

From his general study of the conditions, including especially the topography, the engineer must decide whether the system of sewerage shall be a *separate* system, or a *combined* system (see Articles 10 to 13, inclusive).

The question of the *outlet* will be one of the most important controlling points to be decided, and the engineer must carefully examine all possibilities in this line. The *number of outlets should be as small as feasible, one outlet being secured if possible.* The outlet must be low enough to drain thoroughly all portions of the district it serves, and should be chosen with a view to safe and satisfactory disposal of the sewage.

Sewage disposal is one of the very important points to be con-

sidered. In the past, most cities have simply discharged their sewage into the nearest available body or stream of water which it was considered could be used without causing damage or injunction suits on account of the pollution. At the present time, cities are being compelled more and more to provide means for purifying the sewage (see Articles 110 to 124); and the engineer, in choosing the outlet and planning the sewers, should always consider it probable that in the not distant future the city will be compelled to use some method of purification, and his plans should be so made as readily to permit this in the future, even if the city builds no sewage purification works at first.

During the reconnaissance, the engineer must constantly be recording the significant information he secures, in a neat and systematic manner in *a standard notebook,* which he keeps for the purpose. *Loose-leaf notebooks* of pocket size have many advantages for this purpose. In the same notebook, he should make all his preliminary computations.

On completing the reconnaissance, the engineer usually makes a *preliminary report* to the city officers, stating the conditions he has found, and his conclusions as to the general features of the system he has decided to recommend as best. He also usually presents at this time some rough estimates of cost.

The city then decides whether or not to adopt the general recommendations of the engineer, and whether to go on with the preparation of plans and specifications.

91. Surveys for Sewer Plans. After the reconnaissance, if it is decided to go ahead with the plans, the next step will be to make the necessary *surveys.* These may usually be divided into three principal parts as follows:

(1) *Surveys of Sewage Disposal Site.* In case a sewage disposal plant is to be built, a survey of the site must be made to secure the data needed for the design. Usually this will include data for a *contour map* of the entire tract, and borings or pits to determine the character of the soil.

(2) *Surveys for the Outlet Sewer.* Transit and level lines must be run, and profiles prepared, to determine the best route for the outlet sewer. Data must be secured for an accurate map and profile of the final location of this sewer.

(3) *Surveys for the Street Sewers.* Usually, existing plats can be found sufficiently accurate to give the dimensions necessary for constructing the *general sewerage map*, without special surveys. Small errors on these plats will not affect the general design, and will not be of much importance in view of the accurate surveys which must be made later during construction. Sometimes a few measurements with tape-line and transit must be taken in special localities. Usually the main part of the surveys for the street sewers consists in running *lines of levels* along all the streets on which there is possibility of planning sewers, in order to secure the data necessary to make the *sewer profiles* of all the sewers.

These levels should be referred to the *city datum*—that is, the reference level above which all city elevations are given. If such a datum has not already been adopted, one should be established, and marked by a *permanent bench-mark.* A six-inch iron pipe set six feet in the ground, filled and surrounded with concrete, makes a good, permanent bench-mark. The top, not quite filled with concrete, projects a little above the ground, and a copper bolt is set in the concrete at the top, the top of the bolt constituting the bench-mark. The pipe should have a hinged iron cap to protect the bolt.

In running the level, no effort should be made to trace out the main lines of sewers and their branches, but *each street should be surveyed by itself.* A zero point should be taken at some definite point (such as the center line, or one of the side lines, of a cross-street) at one end of the street, and *station points* 100 feet apart determined by continuous measurements with a steel tape. These stations should be numbered continuously from the zero point, intermediate points being located, in the usual way, by *plus* distances from the preceding station. Thus station 9 + 72 is 972 feet from the zero point.

The exact plus of each side line of each cross-street, and of points opposite other important things, should be determined and recorded in the notebook, to give measurements to be used in preparing the profiles, and in checking the map.

All lines of levels must be checked. At the end of each street, the leveling can be extended across to an adjacent street, and checked with the line of levels on that street.

Numerous bench-marks should be established around the city,

located on permanent points, such as the tops of the foundation walls of buildings.

92. Sewerage Plans. From the data obtained by the surveys, the sewerage plans must be prepared. These will usually consist of a large number of separate sheets, the following being a list of the sheets of one particular set of plans, for a separate system of pipe sewers.

1. Index Sheet. (Giving the contents of all other sheets.)
2. General Sewerage Map.
3. General Map of Sewage-Disposal Plant.
4. Detailed Plans of Septic Tank. (For the Sewage-Disposal Plant.)
5. Detailed Plans of Filter Beds. (For the Sewage-Disposal Plant.)
6. Plans of Standard and Drop Manholes, and Lampholes.
7. Plans of Combined Manholes and Flush-Tanks.

8 to 33. Profile Sheets. (Showing profiles of all the sewers.)

In other cases, separate sheets may be needed for many other things, as, for example,

Details of Brick Sewers, of different sizes.
" " Concrete Sewers, " "
Plans of Flush-Tanks.
" " Catch-Basins.
" " Street Inlets.
" " Sewage Pumping Station.
Etc., etc.

For the sake of convenience and of neatness and system, *all the sheets of a set of sewerage plans should be made of a standard size (one or two can be made larger and folded to the standard size), and they should be bound together in regular book covers,* 18 inches by 24 inches being a convenient standard size of sheet for most cases.

Fig. 37 is a photographic view of such a cover containing a set of sewerage plans. The cover protects the sheets from injury, and is so arranged that any sheet can readily be removed and replaced A cover like that shown costs about $1.50.

The original drawings were all made on tracing cloth, except the profiles, which were made on transparent profile paper. Thus all the sheets can readily be reproduced by the process of blue-printing, and only the blue-print sheets are used on the work or by the City, the engineer retaining the original tracings in his office, where they can be kept safe.

In such a set of plans, the sheets should be numbered in order (see Figs. 38 and 39); and a *standard title* (see title of Fig. 38) should

be adopted for all sheets which will require few changes of the different sheets.

Sewerage Map. In Fig. 38 is shown a reduced copy of an actual *sewerage map* of a separate system of sewers for a small town. The original size of the map shown was 36 inches by 24 inches, so that folding it once reduced it to the 18-inch by 24-inch size.

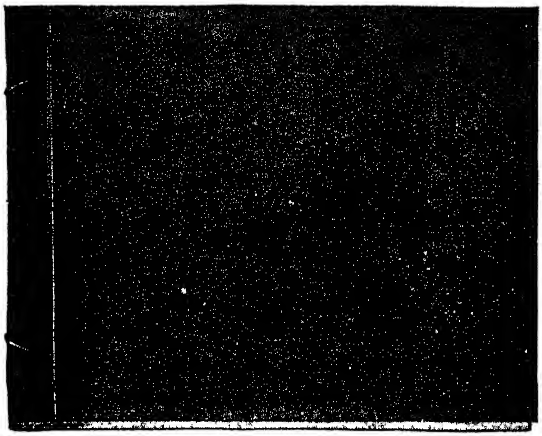


Fig. 37. Standard Cover for Sewerage Plans.

The original scale of the map shown was 200 feet per inch; but for larger places, 300 feet or even 400 feet per inch may be sufficient, since large-scale maps of all the individual sewers appear on the profile sheets.

The lines of sewers in a system such as shown in Fig. 38, ought to be restricted as far as possible to the streets on which the lots front. Sewers on cross-streets add to the mileage of sewers without serving additional lots, and are useless except for connecting other sewers.

The manholes, lampholes, flush-tanks, etc., should be numbered systematically, something as shown in Fig. 38, no two structures of the same kind having the same number. This avoids danger of duplication where the same structure is shown on two or more sheets, as is often the case.

Sewer Profiles. In Fig. 39 is shown a sample profile sheet from an actual set of plans.

The original profile was made on "Plate B" transparent profile paper, so that the profiles can be reproduced easily by blue-printing, the same as the other drawings. The sheets were cut to the standard size, 18 inches by 24 inches, to bind with the other drawings.

The profiles should be made in systematic order of the streets, each

street completed before beginning the next, instead of trying to follow up the main lines of the sewers and their branches.

The profile sheets show large-scale maps of the individual

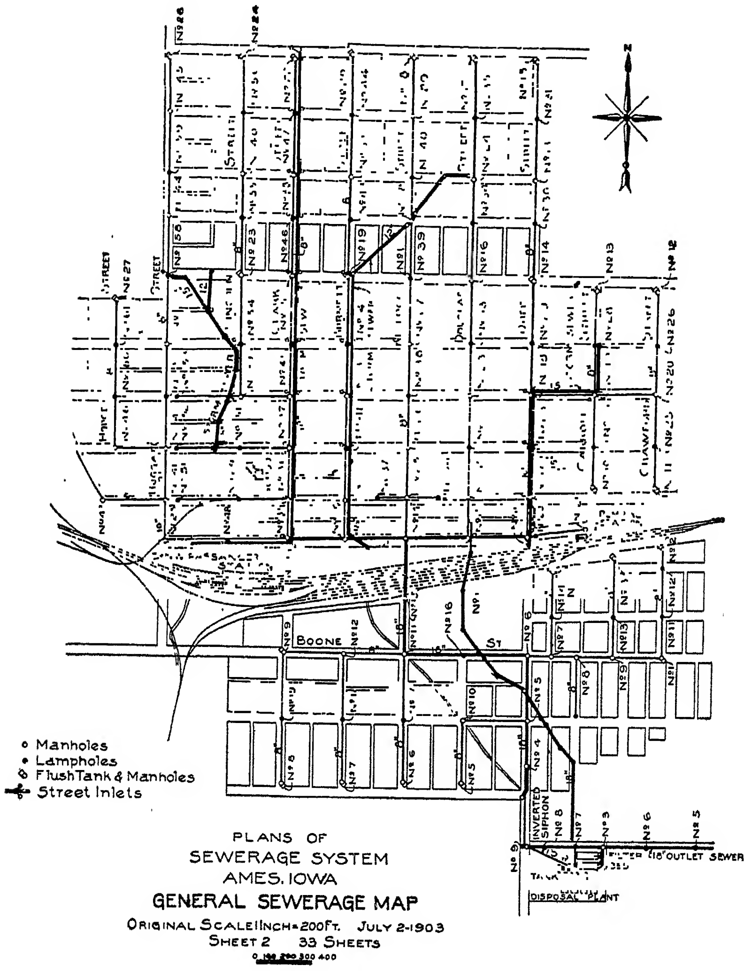


Fig. 38.

sewers immediately below their profiles, to permit the exact location of manholes, etc., and of the sewer itself in the street.

93. Specifications for Sewers. Besides the plans, it will be necessary for the sewerage engineer to prepare precise instructions regarding all matters of importance not fully shown by the plans,

likely to come up during the construction of any part of the sewerage system. Such instructions are called *Specifications.*

An ordinary set of sewer specifications will consist of three parts:

(1) A *Notice to Contractors*, or form of advertisement for the city officers, to use in advertising for bids.

(2) A *Form for Proposal*, with suitable blanks, on copies of which, furnished by the city, all contractors are required to make their bids.

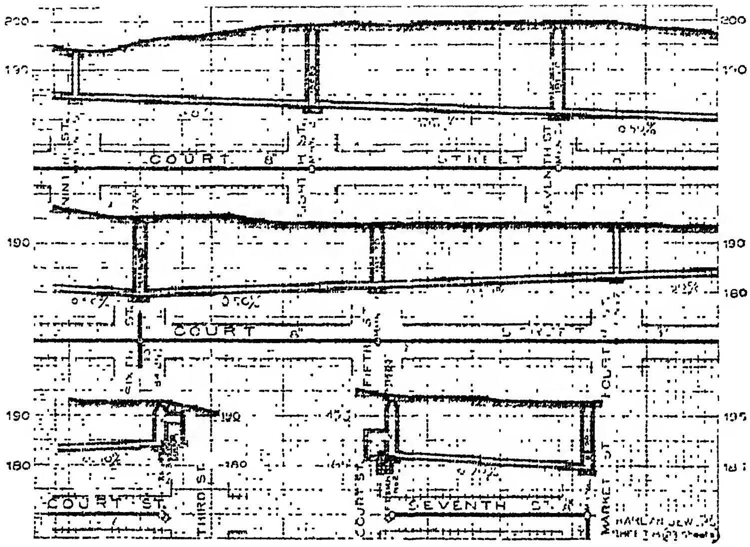


Fig. 39. Typical Sewer Profile Sheet.

(3) *The Specifications Proper.* These again will consist of two main divisions:

(*a*) General clauses, relating to payments, guarantees, etc., and to general features of the work.

(*b*) Specific clauses, specifying the exact details of different parts ot the work.

A copy of an actual set of specifications for the construction of a separate system of pipe sewers, with a sewage-disposal plant, is given herewith:

CITY OF____________________, ________

SPECIFICATIONS

FOR

SEWERS AND SEWAGE-DISPOSAL PLANT

NOTICE TO CONTRACTORS

The Incorporated City of——————,———., will receive

sealed bids until———, ———, at ————————; (1) for the construction of a sewage-disposal plant, consisting of a sewage tank of about ———gals. capacity, and —— sand filter beds, each of about——sq. ft. area; and (2) for the construction of sewers as follows: about —— ft. of 18-inch, —— ft. of 15-inch, —— ft. of 12-inch, —— ft. of 10-inch, and —— ft. of 8-inch, with suitable appurtenances, all in accordance with plans and specifications prepared by ——————, Engineer, ——————, and now on file in his office and with the City Clerk. All bids must be accompanied with certified checks, approximately in the amount of 5 per cent of the bid, made payable without recourse to the City of——————,———. The City reserves the right to reject any or all bids, to waive defects, and to accept any bid. All bids must be in sealed envelopes, marked on the outside "Sewerage Bids," and addressed to———————, City Clerk.

INSTRUCTIONS TO BIDDERS, AND GENERAL SPECIFICATIONS

(1) *Items.* The items of work intended to be covered by these specifications are those required for the entire completion of the System of Sanitary Sewers for the City of——————————, ——————, according to the plans prepared by——————, Engineer, and include the following:

(*a*) The construction of a Sewage-Disposal Plant, including a sewage tank of about ——— gallons capacity, and —— sand filter beds, each of about —— sq. ft. area, and including all valves, sewer pipes, outlets, etc.

(*b*) The construction of Sewers as follows:

18-inch.........................	Ft.
15-inch.........................	"
12-inch.........................	"
10-inch.........................	"
8-inch.........................	"
Manholes	"
Lampholes......................	"
Combined Manholes and Flush-Tanks,	"

together with subdrains as directed by the City.

(2) *Application.* These general specifications and instructions to bidders shall apply to all items of workmanship or materials enumerated above or hereinafter mentioned.

(3) *Definitions of Terms.* Wherever the word "City" is used in these specifications, it shall be understood to mean the Incorporated City of———, ———., acting through the Mayor and Council, or their duly authorized representatives. Wherever the word "Contractor" is used in these specifications, it shall be understood to mean the person or firm employed to do all or any part of the work or furnish all or any part of the material for the Sanitary Sewerage System. Wherever the word "Engineer" is used in these specifications, it shall be understood to mean the Engineer employed by the City to design or supervise the construction of all or any part of the Sanitary Sewerage System.

(4) *Bids.* All bids must be on blanks furnished by the City for the purpose. The blanks can be obtained from ————————, City Clerk ——————, ———., or from —————, Engineer, ————. ———.

All bids must be enclosed in sealed envelopes addressed to———————, City Clerk,————————, ——————., and plainly marked on the outside with the words "Sewerage Bids."

Each bid must be accompanied with a certified check approximately in the sum of 5 per cent of the bid, and made payable without recourse to the City Treasurer, ——————————,————————.

The City reserves the right to reject any or all bids, to waive defects, and to accept any bid.

(5) *Certified Checks.* The certified check mentioned above will be forfeited as damages to the Incorporated City of ——————————, ————————., unless the Contractor enters into contract and furnishes bonds satisfactory to the Mayor and Council within 12 days after the contract has been awarded to him. Certified checks not so forfeited shall be returned to the bidders as soon as the contract is signed and satisfactory bonds are furnished.

(6) *Bond.* A bond satisfactory to the Mayor and Council shall be furnished by the Contractor, approximately in the amount of 50 per cent of the contract price.

(7) *Time.* The Contractor shall begin work within 3 weeks after the contract is awarded to him, and shall entirely complete the work on or before ——————————————, ——————————.

(8) *Sub-contracts.* No sub-contracts shall be awarded to parties unacceptable to the City.

(9) *Progress of the Work.* The work shall be prosecuted at a rate to enable its completion within the time specified; and should the Contractor fail to do this, the City may, after giving ten days' written notice, take over the work and complete it at the Contractor's expense.

(10) *Penalties.* Should the Contractor fail to complete the work at the time specified, he shall forfeit to the City a sum equal to all damages to it resulting from the failure to complete the work at the time specified.

(11) *Delays.* No claims for damages shall be made against the City on account of delays in delivery of materials or performance of work; but should there be unduly prolonged delays in the delivery of any materials or the performance of work on the part of the City, the Contractor shall be entitled to corresponding extension of time.

(12) *Obstructions.* The Contractor shall carry on the work in such a way as to obstruct the city streets as little as possible, and so as not at any time entirely to shut off passage of teams and pedestrians at any place. He shall provide temporary crossings satisfactory to the City for this purpose wherever necessary.

(13) *Precautions.* The Contractor shall take all necessary precautions to prevent injury to the public or to his workmen or to stock, such as providing crossing plank, fencing off his work, keeping lanterns burning at night, etc. He shall hold the City harmless against all claims for damages.

(14) *Plans and Specifications.* The City's plans and these specifications shall be a part of the contract, and all materials and workmanship shall be in accordance with them.

(15) *Supervision.* All materials and workmanship shall be subject to the supervision and inspection of the City and of its Engineer or other authorized representative. Instructions as to the details of the work shall be carried

out, and rejected materials and work shall be promptly removed at any time discovered.

(16) *Quality of Materials and Workmanship.* All workmanship and materials shall be of the best quality.

(17) *Quantities.* The quantities named in the notice to contractors, the form of proposal, or in these specifications, are approximate only. The City shall have the right to vary them; and, if so varied, the total contract price shall be increased or diminished at the rates named per unit in the contract.

(18) *Extra Work.* No extra work shall be done without written orders from the City or its specially authorized representatives placed in charge of the work. In case extra work becomes necessary, it shall be done by the Contractor if so ordered, and shall be paid for by the City on the basis of actual cost, *plus* 10 per cent; but no extra work will be paid for unless ordered in writing by the proper authority at the time undertaken.

(19) *Changes in Plans.* The City shall have the right to make changes in plans. In making such changes, the unit prices named in the contract shall be used, as far as possible, in calculating the changes in price on account of changes in the plans, and where these do not apply, the changes in price, unless a special agreement between the City and the Contractor as to prices is made at the time the changes are ordered, shall be calculated on the same basis as extra work.

(20) *Claims.* The Contractor shall guarantee the payment of all just claims for materials or labor in connection with his contract. Preliminary to the payment for any work, he shall, if required by the City, present evidence satisfactory to the Mayor and Council that all bills for materials and labor have been paid, and any or all payments may be reserved until such evidence has been presented. If the payment of any just claim shall be deferred more than four weeks after written notice has been given concerning it to the Contractor, the City may proceed to pay such claim out of any money due the Contractor.

(21) *Payments.* Payments shall be made as follows:

(NOTE: Fill in, in this blank, whether the payment is to be made in cash, in sewer warrants, sewer certificates, or otherwise. Also whether payments are to be made monthly as the work progresses, or reserved until completion, the former plan being usual for cash payments, and the latter for payments in certificates.)

All payments shall be on estimates prepared by the Engineer and approved by the Council, of materials delivered and work performed; and in case of all payments made prior to the completion of the contract, 15 per cent of the estimate shall be reserved until the final payment on completion of the work.

No payment shall be considered as releasing the Contractor from obligation to remove and make good defective work and materials when discovered at any time.

Two per cent of the total cost may be reserved by the City for one year after the completion of the work, and any part of this reserve may be used to make good defects developed within that time from faulty workmanship and materials, provided that notice shall first be given the Contractor, and that he may promptly make good such defects himself if he desires.

(22) *Guarantee.* The Contractor shall guarantee the workmanship and materials for one year, and keep the system in repair after completion, as provided in clause 21 above.

(23) *Risks.* All materials and work will be at the risk of the Contractor until the final acceptance of the same.

(24) *Cleaning Up.* On completion of each part of the work, all rubbish and unsightly materials must be removed and disposed of as directed by the City, and the streets and grounds left in neat condition. For the sewers, each two blocks must be cleaned up immediately on completion, and on the completion of the entire contract shall be further put in good shape if needed.

MATERIALS

(25) *Vitrified Sewer Pipe.* All sewers shall, unless special permission be given to use cement sewer pipe, be constructed of first-quality salt-glazed, vitrified clay sewer pipe, of the hub-and-spigot pattern, of standard thicknesses and dimensions of hubs. The dimensions of hubs shall be sufficient to leave an annular space for cement of at least ⅜-inch thickness for 8-inch and 10-inch pipe, and ½-inch thickness for larger diameters.

Pipe may be furnished in lengths of 2, 2½, or 3 feet. All pipe and specials shall be sound and well burned, with a clear ring, well glazed and smooth on the inside, and free from broken blisters, lumps, or flakes which are thicker than ⅛ the nominal thickness of the pipe and whose largest diameters are greater than ⅛ the inner diameter of said pipe; and the pipe and specials having broken blisters, lumps, and flakes of any size shall be rejected unless the pipe can be so laid as to bring all of these defects in the top half of the sewer. No pipe having unbroken blisters more than ⅛ inch high shall be used, unless these blisters can be placed in the top half of the sewer. Pipes or specials having fire-checks or cracks of any kind extending through the thickness shall be rejected.

No pipe shall be used which, designed to be straight, varies from a straight line more than ⅛ inch per foot of length; nor shall there be any variation between any two diameters of a pipe greater than $\frac{1}{24}$ the nominal diameter.

No pipe shall be used which has a piece broken from the spigot end deeper than 1½ inches or longer at any point than ¼ the diameter of the pipe; nor which has a piece broken from the bell end if the fracture extends into the body of the pipe, or if such fracture cannot be placed at the top of the sewer. Any pipe or special which betrays in any manner a want of thorough vitrification or fusion, or the use of improper or insufficient materials or methods in its manufacture, shall be rejected.

(26) *Sewer-Pipe Specials.* All T- and Y- junction curves, etc., required shall be furnished and set without extra charge, and shall conform to the pipe specifications as to quality. Y's for house connections may be required every 25 feet on the average, and shall be closed by vitrified stoppers cemented over sand.

(27) *Drain-Tile.* All drain-tile shall be best-quality vitrified agricultural drain-tile in one-foot lengths. All junctions and inspection openings shall be made with suitable T- and Y- junctions and curves, furnished and set without extra charge.

(28) *Brick.* All brick used on the work shall be sound, partially vitrified, well-shaped brick, equal to No. 2 paving brick.

(29) *Cement.* All cement used shall be ——, ——, ——, ——, ——, ——, or —— Portland Cement, perfectly fresh, and not damaged in any particular. It shall be subject to the Standard specifications of the American Society for Testing Materials, and will be rejected if it does not meet these requirements. All cement shall also be subject to close inspection as it is used on the work, and damaged cement will be rejected and must be promptly removed.

(30) *Sand.* All sand shall be clean, sharp, and coarse. All sand for mortar for sewer joints or brick masonry must have all pebbles screened out.

(31) *Broken Stone and Pebbles.* The aggregate for concrete shall consist of either broken stone or screened pebbles passing a 2½-inch ring for ordinary concrete, and a 1½-inch ring for the septic tank. The materials must be sound and hard and durable. The sand must be screened out of pebbles used; but the fine materials need not be screened out from broken stone, a reduction being made in the amount of sand used, approximately equal to the amount of stone dust.

(32) *Cast Iron.* All cast iron shall be good, tough, gray iron, free from defects. Castings shall be smooth and free from blowholes or other flaws.

(33) *Cast-Iron Water-Pipe.* All cast-iron pipe shall be cast of the hub-and-spigot pattern, of standard weights for water-pipe for light pressures. The pipe shall be well coated.

(34) *Valves.* All valves shall be iron body, brass-mounted, hub-end, double-gate, water valves, well coated, of the ———— or of equal make acceptable to the Engineer.

(35) *Valve Boxes.* All valve boxes shall be ———— extension boxes with 5¼-inch shafts, or some equal make acceptable to the Engineer.

MORTAR AND CONCRETE

(36) *Mortar.* All mortar for brickwork or other masonry shall be made of one part of Portland cement to three parts of sand; and all mortar for sewer joints, of one part of cement to one of sand, both ingredients being measured loose and thoroughly mixed. All mortar shall be mixed fresh as used, and any mortar which has begun to set shall be thrown away and not used at all on the work.

(37) *Concrete.* All masonry shown on the plans to be made of concrete shall be constructed with Portland cement, sand, and either broken stone or screened pebbles passing a 2½-inch ring, in the proportions 1-3-5 for ordinary work, and 1-2-3½ for the septic tank, the cement being measured packed as it comes in sacks or barrels, and the sand being measured loose as thrown into the measuring box with shovels. The proportions shall be determined by suitable measuring boxes, or by the use of wheelbarrows. In case of hand-mixing, the sand and cement shall first be thoroughly mixed dry until the color of the mixture is uniform. They shall then again be mixed with water, and then again with the freshly wet aggregate, each mixing being very thorough, and sufficient to secure perfect mixture of the materials. If a machine mixer is used, it shall be of a make acceptable to the Engineer, and shall be so used as to give very thorough mixing. Just enough water shall be

used to make the concrete slightly quake when thoroughly rammed, the water freely flushing to the surface under the ramming.

In depositing, the material shall be deposited in layers not exceeding 6 inches in height, and thoroughly rammed. Where work is left for the night, the layers shall be racked back. Where fresh concrete is deposited on work which is already set or begun to set, the surface shall first be thoroughly cleaned and wet, and washed with a coat of liquid neat cement. After the concrete is deposited, great care shall be taken not to disturb it until the work is thoroughly set. The work shall be protected from the sun, and shall be wet from time to time, until it is thoroughly set.

TRENCHING, PIPE-LAYING, REFILLING, ETC.

(38) *Excavation.* The excavation shall be made exactly to line and grade as indicated by stakes set by the Engineer. At the bottom, the trench shall have a clear width at least one foot greater than the external diameter of the body of the pipe. The last four inches shall be excavated only a few feet in advance of the pipe-laying, by men especially skilled, measuring from an overhead line set parallel to the grade line of the sewer. The bottom of the trench shall be rounded to fit the pipe; and holes shall be dug for the bells so as to give a uniform bearing, and permit the proper construction of the sewer joints on the under side of the pipe. The earth taken from the trench shall be deposited neatly at the sides, in such manner as to obstruct the streets as little as possible; and a clear space of two feet next the trench shall be left on the side on which the Engineer places his stakes. Great care shall be taken to preserve and not to cover up the Engineer's stakes.

(39) *Sheathing.* Wherever necessary to prevent caving of the banks or injury to adjacent pipes or buildings, the Contractor shall, at his own expense, brace and sheath the trenches sufficiently to overcome the difficulty to the satisfaction of the Engineer. If such bracing and sheathing is left permanently in the trench by order of the Engineer, it shall, on refilling, be cut off one foot below the surface and shall be paid for by the City at the price named in the contract; but otherwise the Contractor will receive no extra compensation for it.

(40) *Water in Trenches.* In general, all water encountered in trenches must be drained away through the sub-drains or pumped or bailed out, and the trench must be kept dry for the pipe-laying. In no case shall the sewers be used as drains for such water, and the ends of the sewer shall be kept properly blocked during construction. All necessary precautions shall be taken by the Contractor to prevent the entrance of mud, sand, or other obstructing material into the sewers or subdrains; and on completion of the work, any such materials which may have entered must be cleaned out and the sewers and subdrains left clean and unobstructed.

(41) *Refilling.* In refilling, earth free from stones shall be carefully placed by hand under and around the pipe and to the height of two feet above the top of the sewer, and thoroughly and carefully rammed in layers of not more than six inches' depth.

The remainder of the refilling shall be carefully done. Scrapers may be used if desired. The refilling shall be thoroughly flooded by the Contractor according to the direction of the Engineer, the City furnishing the water free

at the hydrant; but the refilling shall be carried on in such a way that water is taken only as directed by the Waterworks Superintendent, and so that not more than —— gallons of water shall be required in any one day.

Where the trench is not flooded, it shall be left neatly rounded off on top to a height of twice as many inches as the top width of the trench in feet; and the City may from the 2 per cent reserve make good any settlement below the street surface within one year from the date of completion, notice being first given the Contractor, who may promptly do the work himself if he desires.

All surplus material shall be removed to such point within the limits of the sewer district as may be designated by the City; and in case of deficiency of material, it shall be supplied by the Contractor. The street surface shall be left in neat, sightly condition.

(42) *Foundations.* In case the material encountered should be such as not to be suitable for foundations for the sewer, the Engineer shall direct the character of foundations to be constructed, and this shall be paid for by the City as extra work.

(43) *Protection to Buildings.* The Contractor shall take all necessary precautions to protect building and other structures adjacent to the sewer trenches from injury on account of his work, and shall be responsible for all damages to such structures.

(44) *Existing Sewer and Water Mains.* Wherever existing sewers or water mains are encountered in the work, all necessary precautions shall be taken to prevent injury to them; and in case of an injury, it shall be made good by the Contractor without additional compensation. In case any sewer, drain, or water main should be encountered whose present grade should require changing on account of the new sewers, the work necessary for this shall be performed by the Contractor according to the directions of the Engineer, and shall be paid for as extra work.

(45) *Pipe-Laying.* In pipe-laying, each piece must be set exactly to grade by measuring from the invert to a tightly stretched cord set parallel to the grade line, according to stakes or marks given by the Engineer, and supported at least every 25 feet. In making each joint, a gasket of oakum or hemp freshly dipped in cement grout must first be used and packed into place, so as to make the inverts match exactly, giving a smooth, true flow-line. The joints shall afterwards be tightly packed full and beveled off with 1 to 1 Portland cement mortar; but the cementing must be done at least two pipe lengths behind the pipe-laying. The bell-holes must then be immediately packed with sand to hold the cement in place. Great care must be taken to leave no projecting cement or strings of gaskets on the inside of the sewer, and to make all joints as nearly water-tight as possible. Especial care must be taken in forming the joint on the under side of the pipe.

(46) *House Connections.* At points indicated by the Engineer opposite each lot, and at such other points as may be indicated by the Engineer, 4-inch Y's shall be laid, with the branch tilted up at an angle of about 45°. These shall be furnished and laid without extra charge, up to an average of one in each 25 feet.

At points indicated by the Engineer, deep-cut house connections shall be put in according to the plans. The City shall pay for these the regular contract price.

In both ordinary and deep-cut house connections, the connection shall be closed by a vitrified stopper filled over with sand and lightly cemented.

(47) *Subdrains.* Wherever directed by the City, drain-tile subdrains of diameters directed by the Engineer shall be constructed. Each drain shall be laid just at one side of the sewer, at a depth below the sewer invert equal to the external diameter of the subdrain, *plus* three inches. Each joint shall be wrapped twice with a 4-inch strip of muslin at the time laid. The subdrains shall be laid carefully to line and grade; and wherever the Engineer may direct, 4-inch Y's stopped with brick shall be placed. In general, these Y's will be placed at the same points as the house connections on the sewer.

(48) *Subdrain Outlets.* Wherever directed by the Engineer, subdrain outlets shall be constructed, also as directed by the Engineer, and shall be paid for by the City on the basis of cost as determined by the Engineer, *plus* 10 per cent.

(49) *Measurements.* All measurements of sewers, subdrains, etc., shall be in horizontal lines from center to center of manholes and junctions.

MANHOLES AND OTHER APPURTENANCES

(50) *Manholes.* Manholes shall be constructed as shown on the plans and provided in these specifications, the exact location being indicated by the Engineer. All joints in the brickwork shall be shove joints, being filled full. Especial care shall be taken in forming the channels in the concrete bottoms, and wooden templates or half-sewer-pipe shall be used for this work, as directed by the Engineer. Drop manholes shall be constructed as shown on the plans without additional charge over the price bid, which shall be considered an average price.

(51) *Combined Manholes and Flush-Tanks.* Combined manholes and flush-tanks shall be constructed as shown on the plans and as specified for manholes in clause 50. The siphons shall be carefully set, and the cost of furnishing and setting shall be included in the price bid. The Contractor shall provide and set the water connection and bibbs from a point one foot outside the outside wall, on such side as the Engineer may direct.

(52) *Siphons.* Siphons shall be used as shown on the plans, guaranteed by the manufacturers, and tested after being set before acceptance. For the 8- and 10-inch sewers, 6-inch siphons shall be used, and 8-inch for all sewers larger than 10 inches.

(53) *Lampholes.* Lampholes shall be constructed as shown on the plans and provided in these specifications, the exact locations being indicated by the Engineer. The refilling shall be carefully placed and thoroughly rammed by hand in layers not exceeding 6 inches, around and to a distance of three feet each side of each lamphole. Special pains shall be taken to keep the lampholes truly vertical.

SPECIFICATIONS FOR SEWAGE-DISPOSAL PLANT

(54) *Grading.* All grading shall be done as shown by the plans. The bottom of the filter beds and bottom and sides of the septic tank shall be shaped to true surfaces by hand. All slopes shall be neatly dressed.

Should there be a deficiency of earth for the embankments, the Contractor

may borrow from neatly-shaped borrow pits located on adjacent city land, where directed by the Engineer, leaving a smooth, uniform surface. Should there be surplus material, it shall be deposited along the edge of the lake, as directed by the Engineer.

(55) *Concrete Moulds.* The Contractor shall provide moulds of plank not less than two inches in thickness, thoroughly braced at intervals sufficiently close together to avoid distortion of the moulds. These planks shall be dressed on their edges and on the faces next to the wall. The moulds shall not be removed until the walls have become thoroughly set.

(56) *Facing of Concrete Walls.* In the construction of concrete walls, care shall be taken to keep all pebbles or stones away from the faces of the walls, so that the face shall be smooth and free from cavities or exposed stones or pebbles. The upper surface of the roof shall be floated with 1-2 thin mortar applied when the roof is made, and all cavities in other concrete surfaces filled and smoothed with 1-2 mortar.

(57) *Cement Wash.* On completion of concrete walls and floors, and after removal of the moulds and pointing up defects, all interior surfaces of floors and walls and roof, and the upper surface of the roof, shall be given two good coats of thin, neat Portland cement grout applied with a whitewash brush, time being left between applications for the first coat to set hard.

(58) *Alternating Siphons.* The alternating siphons shall be provided of the make shown on the plans, and set by the Contractor, strictly according to the directions of the manufacturer as given through the Engineer. Any imperfections affecting the working of the siphons when they are tested shall be corrected by the Contractor,who must guarantee their satisfactory working.

(59) *Filters.* The pebbles for the bottoms of the filters shall be screened clean of sand and properly graded, the 2-inch layer of fine pebbles being small enough to hold up the sand placed over it. All sand shall be clean and coarse, but the pebbles need not be screened out. In placing pebbles and sand, care shall be taken not to injure or disturb the drain tile, and the top surface of the sand shall very carefully be made level. Drain tile shall be laid carefully to line and grade.

(60) *Pipe-Laying.* All sewer pipe and cast-iron pipe shall be carefully laid to line and grade, with gaskets and tight joints, all as provided in the regular sewer specifications.

(61) *Sodding.* All earthwork slopes of the tank and filters shall be neatly sodded.

(62) *Bulkheads.* All bulkheads shown on the plans shall be constructed of Portland cement concrete, with moulds, and with care as to facing the same as provided for the concrete work of the septic tank.

(63) *Reinforcing.* The reinforcing shown on the plans is .... corrugated bars of not less than 50,000 lbs. per sq. in. elastic limit; but other forms of bars having equal elastic limit, equal net area, and a mechanical bond acceptable to the Engineer, may be used. The net area of any bars used must be increased to make good any deficiency in the elastic limit.

....

....

....

For brick sewers, the following specifications are suggested by Folwell in his book on *Sewerage:*

"For brick masonry in straight walls or sewers, none but whole, sound brick shall be used. For manholes, flush-tanks, and similar work, a limited number of half-brick may be used, not to exceed ⅓ of the whole in any case. Unless the Engineer direct otherwise, each brick shall be thoroughly wetted immediately before being laid. It shall be laid with a full, close joint of cement mortar on its bed, ends, and side at one operation. In no case is mortar to be slushed in afterward. Special care shall be taken to make the face of the brickwork smooth; and all joints on the interior of a sewer shall be carefully struck with the point of a trowel or pointed to the satisfaction of the Engineer. Where pipe-connections enter a sewer or manhole, "bull's-eyes" shall be constructed by laying rowlock courses of brick around them, the cost of such construction being included in the regular price bid for the sewer or appurtenances. Around pipe more than 15 inches in diameter, 2 rowlock courses shall be laid.

"Brickwork in sewers shall be laid by line, each course perfectly straight and parallel to the axis of the sewer. Joints appearing in the sewer shall in no case exceed ¼ inch in width. Sewers shall conform accurately in section and dimensions to the plans of the same. All inverts and bottom curves shall be worked from templates accurately set; the arches are to be formed upon strong centers accurately and solidly set, and the crowns keyed in full joints of mortar. No centers shall be drawn until the arch masonry has set to the satisfaction of the Engineer, and refilling has progressed up to the crown. They shall be drawn with care, so as not to crack or injure the work. The extrados is to be neatly plastered with cement mortar ½ inch thick, the arches being cleaned and wetted just before plastering. The end of each section of brick sewer shall be toothed or racked back; and before beginning the succeeding section, all loose brick at the end shall be removed and the toothing cleaned of mortar. All brickwork shall be thoroughly bonded, adjacent courses breaking joints at least ¼ the exposed length of the brick.

"If there should be any distortion of the sewer before acceptance, this shall be corrected by tearing down and rebuilding. No local patching will be allowed, but when repairs are necessary a section shall be removed at least 3 feet long and including the entire arch, or the entire sewer if the defect is in the invert. Leakage of ground water into the sewer shall be similarly corrected, unless it can be prevented by calking the joints with oakum saturated in cement, with wooden plugs, or other material acceptable to the Engineer."

FORM OF PROPOSAL

——————————, ————.

To the Mayor and Council of the Incorporated City of——————, ————.

Gentlemen:

—— have carefully examined the plans and read the specifications prepared for your proposed sewage-disposal plant and sanitary sewers by————, Engineer, and —— agree to furnish all the materials and perform all the labor required for the completion of the proposed work for the following prices:

ITEM	APPROXIMATE QUANTITY	UNIT PRICE	TOTAL PRICE
Sewage Disposal Plant, complete......			
Sewers, complete, including Y's, except subdrains, manholes, lampholes, and flush-tanks.			
18-inch.........................			
15-inch.........................			
12-inch.........................			
10-inch.........................			
8-inch.........................			
Subdrains, complete			
10-inch.........................			
8-inch.........................			
6-inch.........................			
Deep-Cut House Connections, complete.			
Manholes, complete................			
Combined Manholes and Flush-Tanks, complete....................			
Lumber Left in Trenches (per M., B. M.)			

All the above shall be strictly in accordance with the plans and specifications.

In case —— bid is accepted,—— agree to begin work within three weeks after the acceptance of —— bid, and to entirely complete the work on or before——————————, ——————.

——— further agree to enter into contract and furnish bond satisfactory to the City Council within 12 days after acceptance of —— bid.

Respectfully submitted,

————————————

————————————

94. Form for Sewerage Contract. Besides plans and specifications, the sewerage Engineer is sometimes called upon to furnish a *Form of Contract* to be signed by the Contractor and the city representatives, though this, more properly, should be the work of the City Attorney. The following simple form of contract has been used successfully with specifications such as those given above:

This Article of Agreement, made this ——— day of ——————— A.D., ——, by and between ——————, of ——————, ———, party of the first part, and the Incorporated City of ———, ———, acting through its Mayor and Council, party of the second part,

WITNESSETH:

The party of the first part agrees to furnish all material and perform all labor required for the entire completion of sanitary sewers, subdrains, and other appurtenances, on streets in the said City of ————, ———, as follows:

(NOTE: In this space place a list of the sewers included in the contracts by streets, giving the sizes on each street of both sewer and subdrain, and the points at which each size begins and ends.)

All the above sewers are to have manholes and other appurtenances as shown by the plans and specifications.

The party of the first part further agrees that all the above labor and materials shall be strictly in accordance with the sewer plans and specifications prepared for the party of the second part by ——————, Engineer, said plans and specifications identified by the signatures of the parties hereto, being hereby made a part of this contract.

The party of the second part agrees to pay to the party of the first part for the above labor and materials, the following prices:

Sewers, complete, except subdrains, manholes, lampholes, and flush-tanks,

24-inch	$	per lin. ft.
20 "		" "
18 "		" "
15 "		" "
12 "		" "
10 "		" "
8 "		" "

Subdrains, complete,

24-inch		" "
18 "		" "
15 "		" "
12 "		" "
10 "		" "
8 "		" "

Manholes, complete	$	each
Lampholes, complete		"
Combined Manholes and Flush-Tanks, complete		"
Flush-Tanks, complete		"
Lumber ordered left in trenches	$	per M., B. M.

The payments shall be made in ————————————————————————— and paid to the party of the first part in accordance with the provisions of the specifications, 2 per cent being reserved for one year to guarantee the work.

IN WITNESS WHEREOF we have hereunto set our hands and seals the date and place first above mentioned.

——————————

Party of the First Part

SEAL

The Incorporated City of ———, by

——————Mayor,

SEAL

——————————

Party of the Second Part

95. Form of Bond for Sewerage Contract. The Contractor for a piece of sewerage work is usually required to furnish to the City a bond, which is frequently for a sum equal to about one-half the

amount of the contract. The simpler the form of the bond, the better. The following form has been used successfully:

BOND

Know all men by these presents, that we,——————————, of —————————— ,———, Principal, and

————————————————————————————————

————————————————————————————————

————————————————————————————————

Sureties

are held and firmly bound to the Incorporated City of ——————, ————, in the penal sum of —————————————— Dollars (———————), lawful money of the United States of America.

Now, the condition of this obligation is that whereas the above-mentioned ————————, of ——————, ——, has entered into contract with the Incorporated City of ———, ——, dated ———., A. D. ——, to furnish all labor and materials required for the entire completion of about ——— feet of sanitary sewers, subdrains, and other appurtenances for the said City of —————, ————, now, if the said ————————, shall well and truly perform all the obligations of his said contract, strictly according to the terms thereof, then shall this bond be null and void, but otherwise it shall be and remain in full force and effect.

————————————
Principal

————————————

————————————
Sureties

CONSTRUCTION OF SEWERS

96. Letting the Sewer Contract. After the plans and specifications have been completed and accepted by the City, the next step will be to let the contract for the work.

First. The work should be advertised, if possible, three or four weeks in advance, in at least two good engineering or trade journals. It must often, by law, be advertised also in at least one local journal. For a form for the advertisement see pages 112 and 113.

Second. On the day and at the hour specified in the advertisements, the City Council meets to open the sealed bids which have been submitted on the blank "forms for proposals" furnished by the City for the purpose.

Third. If the bids are satisfactory, the contract is awarded to the *lowest responsible bidder.*

Fourth. A contract for executing the work in accordance with the plans and specifications, is signed by the Contractor and by the City.

Fifth. The Contractor furnishes a bond satisfactory to the City.

In all these steps, there is need of great care on the part of the city authorities to make sure that all provisions of the law are com-

plied with, and they should be fully advised at all times by a competent attorney.

97. Organization of Engineering Force during Construction of Sewers. It is not common for the Consulting Engineer who prepares the sewerage plans and specifications, to be constantly on the ground or even in the city during construction. He makes only occasional visits for inspection and consultation.

The actual work of sewer construction is usually directly supervised either by the City Engineer, or by a *Resident Engineer* employed especially for this purpose.

It will be necessary for the resident engineer in charge of the construction of a sewerage system of some magnitude, to have an office and an adequate equipment of drafting apparatus, surveying instruments, etc. He will have employed under him:

Draftsmen and clerks, in the office.
Instrument men and rodmen, to do the surveying.
Inspectors, constantly on all work, to insure its being properly executed.

The resident engineer himself will supervise these employees, visit all parts of the work frequently, and constantly exercise general supervision over all its features.

98. Laying Out the Sewer Work. After checking up the benchmarks on the original survey, it will be necessary for the engineering force to stake out the sewers, keeping somewhat in advance of the actual construction.

The stakes are usually placed a uniform distance to one side of the true line, so as not to be disturbed by the digging of the trench. This distance, and the side on which the stakes are placed, should be the same for all parts of the work, to avoid confusion and mistakes.

The stakes should usually be set about 25 feet apart.

The manholes should usually be located first, in accordance with the profile sheets; and the sewers should be run as straight lines, center to center of adjacent manholes. All discrepancies from the original measurements should each be adjusted, if possible, between the two manholes between which each was found; and such discrepancies should not be carried on to affect all the rest of the work.

There are two methods of giving grades for sewers.

(1) The best method is to set the grade stakes nearly flush with the surface, at a uniform offset to one side of the trench, ascertaining

the distance of the top of each stake above grade by carefully checked levels. By measuring from these stakes, a grade cord, supported on cross-frames every 25 feet, is stretched parallel to the grade line of the sewer, over its center line. For this method of giving grades, see Fig. 40.

(2) Another method is to set grade stakes at the bottom of the trench. This method is adapted only to very large sewers.

99. Trenching and Refilling. Sewer trenching and refilling may be done either by machines or by hand. *Excavating Machines* for sewers are of two types:

(1) *Machines which themselves do the excavating.* These are just coming into use, and are becoming more and more successful.

(2) *Machines which simply carry away the excavated material,* usually dumping it over the completed sewer further back. This type has the advantage of not piling up the dirt in the busy street. It carries, on overhead cableways or trestles, buckets which can be lowered into the trench, and in which the excavated material is placed by hand.

Machines of both types are suited best to comparatively extensive work; and under favorable conditions they lessen the cost materially.

Most sewer trenching, however, is done by *hand.* For such work the men are organized in gangs, the number of men in each gang varying from 20 to 80. Each gang has a foreman, and a water boy, and sometimes a sub-foreman. A pair of pipe-layers may work with each gang, or, if the trench be deep, one pair of pipe-layers may work part of the time with one gang and part with another.

The details of sewer trenching and refilling as ordinarily carried out, are specified quite fully in clauses 38, 40, and 41 of the sample sewer specifications given in Art. 93 (which clauses now read carefully). All details there specified should be enforced by the Inspector and the Engineer.

In clause 41, Art. 93, referred to above, the method specified for compacting the refilling is by flooding with water. While this is the cheapest method, where the water is available, and while it gives good results if properly done, it may be found necessary sometimes, in the case of paved streets, to adopt the more expensive method of *tamping.* For thorough tamping, there should be from 1 to 2 men tamping, to 1 shoveler, and the rammers used should weigh 4 to 6

pounds each. The soil refilled should be moistened if dry, and should be tamped in about 4-inch layers. It is possible by very thorough tamping to compact the soil more thoroughly than by flooding.

100. Sheathing. Except for shallow ditches in very solid earth, it is usually necessary to brace the sides of sewer trenches to prevent their caving in. Such bracing is called *sheathing*. The most common methods of sheathing are illustrated in Fig. 40.

The horizontal members of the sheathing are called *rangers*, and the rangers are held the right distances apart by *sewer braces* of wood or iron. The iron braces are shown in Fig. 40. The rangers are usually about 12 feet long. Behind the rangers are placed the vertical planks of the sheathing, either a few feet apart in firm material, forming *skeleton sheathing*, or in contact with each other in caving material, forming *close sheathing*. The sheathing plank are 2 inches thick and are usually about 10 feet or 12 feet long. The rangers may be 2-inch planks in favorable soil, or 4 by 4 or even 4 by 6 inches in poor soil.

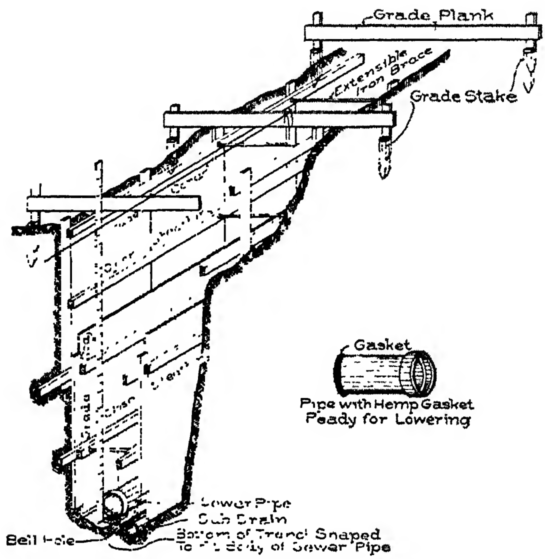


Fig. 40. Diagram Showing Construction of Pipe Sewer.

The sheathing plank are usually driven by hand, with wooden mauls.

Sometimes, for large sewers, heavy *sheet piling* may be driven by pile-drivers, to take the place of ordinary sheathing.

Ordinary sheathing is removed from the trench as the refilling proceeds. In case of special danger to near-by water mains, conduits, or foundations, on account of possibility of the banks caving before the refilling is finally settled, the Engineer may order the sheathing

to be left permanently in the trench. In such case, the Inspector makes record of the exact amount of lumber left in the trench, and the City pays for it.

101. Pipe-Laying. The pipe-laying is usually done by two men, though, with large pipes, another may be needed. These men excavate the last few inches of the trench, as well as lay the pipes.

The laying of every pipe, and the making of every joint, should be carefully watched by an Inspector, who should faithfully enforce the specifications.

For specifications for pipe-laying, see clause 45, Art. 93 (which clause now read carefully).

All the sewer pipe should be carefully inspected before being used, and those pieces rejected which do not meet the specifications. See clause 25, Art. 93. The Inspector should see that no rejected or poor pipe is used.

The Inspector should see that every pipe is laid exactly to grade by measurement from the grade cord (see Fig. 40).

The Inspector should also see that house-connection Y's are placed opposite each lot on each side of the street, at the proper points; and he must exactly locate each such connection by measurements fully recorded in his notebook.

102. Construction of Brick Sewers. For specifications for the construction of brick sewers, see reference to Folwell in Art. 93, p. 122. (Read carefully.)

The construction of a brick sewer is shown in Fig. 41.

It will be the duty of the Inspector to inspect all brick before they are used, rejecting the poor ones, and to fully enforce the specifications for construction. He must also see that the templates are set truly to line and grade, that the house connections are set at the proper places and heights, and accurately located in his records.

In the case of large brick sewers, more trouble is to be expected with foundations than in the case of pipe sewers. Sometimes soft soil or quicksand may make it almost impossible to shape the material in the bottom to fit the outside of circular sewers. In such cases, special foundations, such as shown in Fig. 20, may have to be put in through the treacherous material. Other forms of special foundations are often used.

The Engineer should make full record of all such features of the work.

103. Records of Sewer Construction. *Daily Reports.* The resident Engineer in charge of the construction of a sewerage system, should require, from all members of his engineering force, daily reports, on suitable blank forms, showing the exact work on which each was engaged. Another set of exact reports should show the work accomplished by the Contractor each day, and the materials and labor used on each part of the work.

Data of Sewer Construction. The information from these daily reports should be entered in a permanent book, showing all features

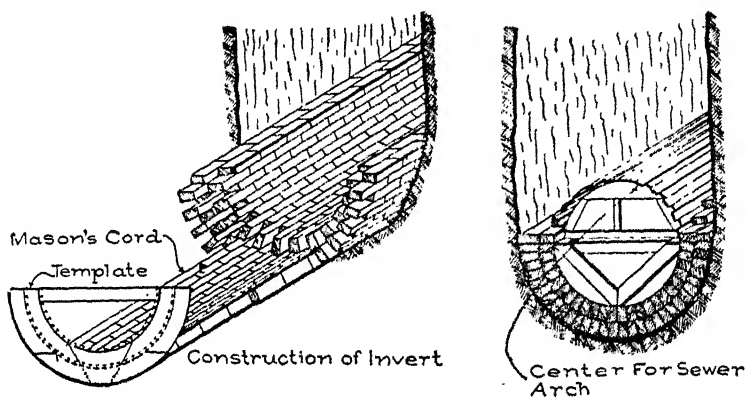


Fig. 41. Diagrams Showing Construction of Brick Sewer.

of the progress of the work, and giving data for itemized estimates of the cost.

Sewer Record Book. In another permanent book, a complete, final record of all the sewers should be entered.

On the left-hand page may be given in order the numbers of the stations of the sewer survey, running from the bottom to the top of the page, together with the surface elevations, the grade elevations, and the rate of grade.

The exact character of the soil should also be shown, with exact levels for computing any rock excavation. Notes should be made of the level and amount of any ground water encountered.

On the right-hand page should be made a large-scale sketch of the sewer, showing its exact location with reference to the street lines

and the lot lines, and the exact location of manholes and other accessories. This sketch should also show the location of all house connections, with exact measurements (such as the station and *plus* of each connection) by which to locate all such connections.

On the right-hand page may also be entered the exact limits of sheathing left in trenches, and the amounts of lumber in such sheathing, as well as the exact limits and character of all special sewer

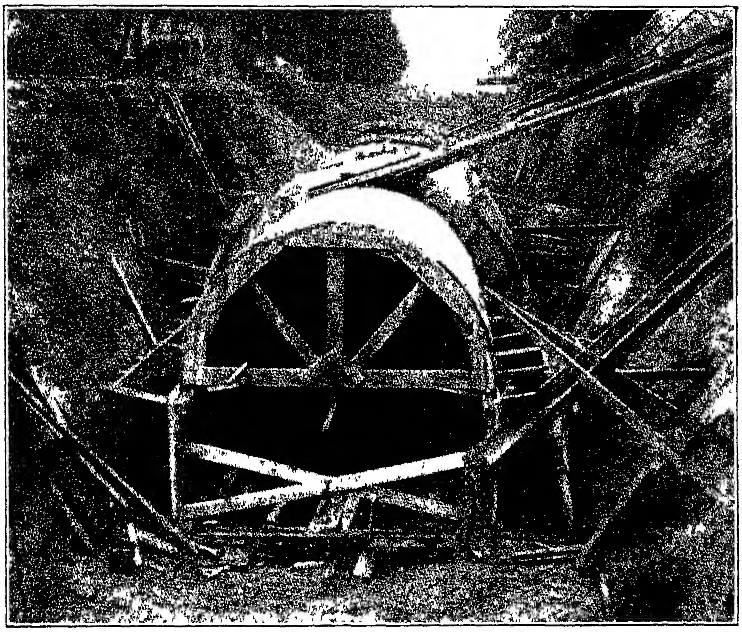


Fig. 42. Construction of Dry-Run Concrete Sewer, Waterloo; Iowa.

foundations, of changes of grade where other conduits are crossed, and of all other extra work.

Final Sewerage Map and Profiles. On completion of the system, the resident Engineer should make a complete final sewerage map, and complete final profiles of all sewers, both corrected by any changes from the original plans adopted during construction.

Plat of Sewer Connections. For small towns, at least, large-scale plats of the different streets should be prepared, showing the exact location of all house connections.

MAINTENANCE OF SEWERS

104. Sewerage Systems should be Carefully Maintained in Good Condition. Too often it appears to be considered that when a sewerage system is completed all further care of it can be neglected with impunity. This is a great mistake. The sewerage system may become a source of danger to the public health, instead of a means of safety, unless it is given proper care and attention.

105. Sewer Ordinances, Permits, and Records. Every city having sewers should pass a carefully prepared *Sewer Ordinance*, prescribing in detail the conditions under which citizens are permitted to use the sewers.

One provision of the Sewer Ordinance should be, that all property owners desiring to make sewer connections shall first secure a *Sewer Permit.* For this and for the application for it, blank forms are provided, which are to be filled in by the applicant, giving full description of the connection. The permit will require the work to be done according to the city regulations.

Every house sewer should be connected with the sewer at a regular house connection. No cutting into the sewer whatever should be permitted, as there is great danger of such cutting ruining the sewer.

Full *Sewer Records* should be kept by the proper city officers, showing full details of all connections with the sewers. This is too often neglected, to the great detriment of the City, which finds itself without means of ascertaining what people or how many are using the sewers, and perhaps putting injurious substances into them.

106. Plumbing Regulations, Tests, and Licenses. The city should also prescribe by ordinance strict *Plumbing Regulations*, setting forth in full detail the requirements for good plumbing (see Articles 76 to 81 inclusive). All property owners should be required to do all plumbing in strict accordance with these regulations.

The work should be carefully *inspected and tested* by a City Inspector, to see that it fully complies with the ordinance. The *water test* is applied by stopping up the outlets of the soil-pipe and of the various fixtures, and filling the pipes with water, when defects will be shown by leaks. In the *smoke test*, the pipes are blown full of smoke; and in the *peppermint test*, oil of peppermint is poured into them. In neither case must it be possible to detect any of the odor in the interior of the house.

Plumbing regulations usually require that plumbing shall be done only by plumbers holding *plumbers' licenses* granted by the City. The proper city officers have blank forms for making applications for such licenses, as well as for the licenses themselves. The plumber making application for a license should be required to show proof of proficiency, and should be placed under bond to comply fully with the sewer ordinance and the plumbing regulations, and to protect the City from damages on account of his work. The plumber may also be made subject to fines for violating the sewer ordinance and regulations, and to revocation of his license.

107. Regular Sewer Inspection. In sewer maintenance, besides the work of granting sewer permits, and inspecting house plumbing and the making of connections with the sewers, the entire sewerage system should be gone over regularly and carefully by a Sewer Inspector, once every two weeks if possible.

The Inspector, in this work, should open all manholes and lampholes, and carefully examine the sewer to make sure that it is keeping clean, well-ventilated, and reasonably free from offensive odors. He should also examine carefully the working of all flush-tanks, to make sure that they are operating satisfactorily. He should also examine all catch-basins, to make sure that they are cleaned frequently enough.

Small defects found on these periodical inspections should be remedied at once, and full notes made of more extensive work found to be necessary.

108. Flushing and Cleaning of Sewers. In many sewerage systems, it is found impossible to prevent absolutely the formation of deposits in the sewers, which must then be removed by hand-flushing, or by direct cleaning of the sewers.

Flushing is ordinarily preferred to hand-cleaning methods where the water for the purpose is available, and where it is readily possible to remove the deposits in this way. For the most common methods of hand-flushing, see Art. 25.

In hand-cleaning, large sewers may be entered by the workmen themselves to remove the deposits. In small sewers, lines are often floated down from one manhole to the next below; and by means of these lines, various cleaning devices are dragged through the sewer, or back and forth in it, to remove the deposits. Sometimes, for small

sewers, a ball, a little smaller than the sewer, with a line attached to haul it back in case of stoppage, is allowed to float down the sewer, from manhole to manhole. The sewage is dammed back by it, and spurts out on all sides under pressure, thus scouring and cleaning the sewer.

For large sewers, discs or gates, traveling on carriages, or boats, may be used, working on the same principle. Many forms of such apparatus have been devised. A notable example of the use on a large scale of traveling sewage-scouring gates is in connection with the Paris sewers, Fig. 24.

109. Cleaning of Catch-Basins. In Art. 27, catch-basins were described; and it was stated that unless they are frequently cleaned they become filled with filth and soil and debris from the street, and fail utterly in their purpose, which is to keep such materials out of the sewers. Moreover—which is still worse than this—uncleaned catch-basins are unsanitary, and are sources of foul odors. Hence catch-basins, when used, should be regularly cleaned, and the City should have a regular arrangement for this work, and should provide labor-saving apparatus for the work, such as hoisting apparatus or special pumps for lifting the material from the catch-basins to the wagons.

SEWAGE DISPOSAL*

110. Basic Principle. The subject of sewage disposal seems prosaic at first glance, but when one considers that its processes involve the whole cycle of life, the study of it becomes most interesting. The organic matters with which we have to deal must be changed from the unstable form in which they exist to stable chemical compounds. This result is obtained by the action of millions of micro-organisms called *bacteria*. "Without them", as Woodhead says, "the surface of the earth would be covered with dead organic matter, the remains of plant and animal bodies, which, retaining the elements necessary for the building up of new plant life and animal bodies, would soon cut off the food supply of new plants and animals; life would be impossible because the work of death would be incomplete," or, as Pasteur puts it, "because the return to the atmosphere and to the mineral kingdom of all that which has ceased to live would be totally suspended."

*The following sections on Sewage and Garbage Disposal have been supplied by Thos. Fleming, Jr., of Chester and Fleming, Hydraulic and Sanitary Engineers, Pittsburgh, Pa.

111. Historical. It was not until the middle of the nineteenth century that sewage disposal was studied or put into effect in a systematic way. Previous to that time, it had been the habit of individuals and communities to dispose of their sewage in a manner the least expensive and yet consistent with preventing a local nuisance, and in most cases where sewer systems had been installed, this resulted in discharging the sewage into the nearest water course. In communities not fortunate enough to have sewer systems, cesspools were abundant, and in many instances were arranged with overflows to the nearest surface drain. In 1858, conditions became so acute in England, with its small streams and large tributary population, that a law was passed prohibiting the pollution of rivers, and during the next few years several able commissions were appointed by the Government to study the problem. The commissions invariably declared that the proper method for purifying sewage was to distribute it on land, although during this period private companies were exploiting chemical processes and endeavoring to have chemical precipitation adopted as the proper form for sewage disposal. The disposal of sewage by broad irrigation was carried out on an extensive scale in England during the latter part of the nineteenth century, and several extensive chemical precipitation plants were also installed. Germany soon followed England in prohibiting the discharge of unpurified sewage into the streams and in adopting broad irrigation and chemical precipitation for treating it. America followed shortly, especially in New England, as there the small streams and dense population made the conditions quite similar to those in England.

In 1886, the State of Massachusetts passed an act preventing river pollution and placing the control of the streams in the hands of the State Board of Health. This Board constructed an experiment station at Lawrence, where extensive tests were made in the use of artificial filters, leading to the construction of the modern biological filters. Numerous experimental stations followed in England and Gemany, which resulted in many permanent installations on a large scale of artificial biological works.

In 1896, the so-called septic tanks were developed quite extensively in England, and, in the last few years, the settling tanks with separate digestion compartments have come into

TABLE XIV

Composition of Sewage

Analyses of Raw Sewage	Parts per Million											
	Suspended Solids			Nitrogen as				Oxygen Consumed				
	Total	Fixed	Volatile	Organic N	Free Ammonia	Nitrite	Nitrate	Total	Suspended	Dissolved	Alkalinity	Chlorine
Atlanta, Ga.	285	138	126	12.8	18.8	0.1	2.2	90.6	...	...	...	...
Columbus, O.	209	130	79	9.0	11.0	0.09	0.20	51.0	25.0	26.0	...	65
Waterbury, Conn.	165	50	115	14.8	7.8	0.14	1.52	46.0	20.0	26.0	41	48
Philadelphia, Pa.	189	59	130	6.3	4.0	0.23	1.00	76.0	35.6	40.4	128	39

prominence together with methods for the disinfection of sewage and the removal of suspended matter by fine screens.

SEWAGE

112. Character of Sewage. The average sewage consists mainly of water used for washing and flushing purposes. In America, the daily water consumption ranges from thirty gallons to four hundred gallons per capita with an average of one hundred gallons per capita. This water, in passing through the sewer system, contains a variable amount of solids and liquids representing the waste products of the community which altogether does not exceed two per cent of the water. If analyzed bacteriologically, it will, however, be found to contain at least one million bacteria per cubic centimeter; and if these bacteria be further differentiated, it will be found that most of them are of a harmless type. The character of sewage is naturally extremely variable, depending on the amount of water consumption per capita, the admission of storm water into the system, the character of effluent from the various industries, and the constituents of the mineral compounds in the water supply itself. It will also vary at different hours of the day.

113. Analyses of Sewage. Table XIV of analyses gives an idea of the composition of sewage. It will be noted that the quantities are expressed in parts per million by weight and that a com-

TABLE XV

Typical Analysis of City Sewage

ANALYSIS OF ALLIANCE, O. SEWAGE JULY 1914
Chester & Fleming Consulting Engineers Pittsburgh, Pa.

	Sewage Flow in Million Gallons	PARTS PER MILLION												Ratio of Available Oxygen to Oxygen Required for Equilibrium Expressed in Per Cent. STABILITY at 37°C				
		SUSPENDED SOLID			DISSOLVED OXYGEN				CONSUMED OXYGEN								
		Screened Sewage	Effluent of Septic Tank #1	" " " #3	Screened Sewage	Effluent of Septic Tank #1	" " " #3	" Contact Beds	Screened Sewage	Effluent of Septic Tank #1	" " " #3	" Contact Beds	Effluent of Septic Tank #1	" " " #3	" Sand Bed #2	" " " #3	" " " #4
July 2	2.51	125	65	90	0	0	0	0	45	24.5	29 2	13 6					
" 3	2.52	190	80	96	0	0	0	0	50	31 6	36 4	16.0					
" 4	2 37	193	105	115	0	0	0	0	79	11.6	27.6	13.6	2	0	33	99	
" 5	2.38	185	96	174	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]				93	93
" 6	2.57	166	116	136	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]					
" 7	2 51	195	35	103	0	0	0	15	84	36 0	47 0	26 8					
" 8	2 45	150	75	135	0	0	0	1 5	49	40 0	42 0	12 8					

plete detailed analysis of the constituent parts of the sewage is not made, but totals of the solids, organic compounds, and other indicative tests are given. The form in which the organic matter exists in sewage is quite complex and would be difficult to analyze, whereas, indicative tests, which can be used for comparison, furnish the information desired.

Table XV shows an analysis of sewage and effluents from the various units of sewage disposal works at Alliance, Ohio. The significance of these tests is as follows:

Suspended Solids. Dr. Imhoff divides solids found in sewage into four classes: (1) settling solids (removed by 2 hours' quiescent sedimentation); (2) finely divided solids (finer than above but removable by filtration through paper); (3) colloidal matter (finer than either of the above but removable by a dialyzing membrane); and (4) solids in true solution.

Of these, the first three are classified as suspended solids and are subdivided into fixed and volatile solids. Most of the fixed solids represent the inorganic matter in the sewage, consisting mainly of the material in the water supply. The volatile solids serve as an index of the amount of organic matter and are the solids with which we have to deal in purification.

Nitrogen. The organic nitrogen indicates the amount of undecayed organic matter containing nitrogen in the sewage, and

the free ammonia indicates the amount of decomposing organic matter containing nitrogen. The nitrites indicate a partial breaking down of the organic matter into inorganic compounds, and the nitrates indicate the amount of organic matter that has been completely broken down and changed into stable inorganic compounds.

Oxygen Consumed. A very important test as indicating the condition of the sewage with respect to stability shows the relative amount of carbonaceous organic matter in the various effluents. It is the amount of oxygen absorbed by the sewage and is used to compare with the effluent as to purification effected.

Alkalinity. The alkaline test furnishes a comparison of the sewage with the water supply, and any serious modification would indicate pollution with trade wastes of an acid character. It also serves as an index of the degree of purification by comparing sewage with effluent.

Chlorine. Chlorine is harmless in the form of common salt, in which it occurs in sewage, and is only indicative of the strength of the sewage.

Bacteria. Bacteria are microscopic organisms belonging to the vegetable kingdom. They have been divided into two classes, namely, saprophytes, which live on inanimate matter, and parasites which live on substances of animal bodies. They are further classified by their ability to thrive in the absence or presence of oxygen. Those which thrive in the presence of oxygen are called *aërobic* bacteria, and those which thrive in the absence of oxygen are called *anaërobic* bacteria. Each of the classes has so-called facultative bacteria also which exist with or without oxygen, but thrive under the conditions suited to their class. Among the subdivisions of these general classes are the pathogenic bacteria which are the means of transmitting water-borne diseases. These bacteria are of course parasites and do not increase after leaving the human body. They may exist, however, for months under most unfavorable conditions.

The reduction of organic matter to inorganic matter is accomplished by the action of enormous numbers of those bacteria upon the organic material. It has been found from experiments and from the results of the operations of numerous biological disposal plants, that *aërobic* bacteria in the presence of free oxygen bring

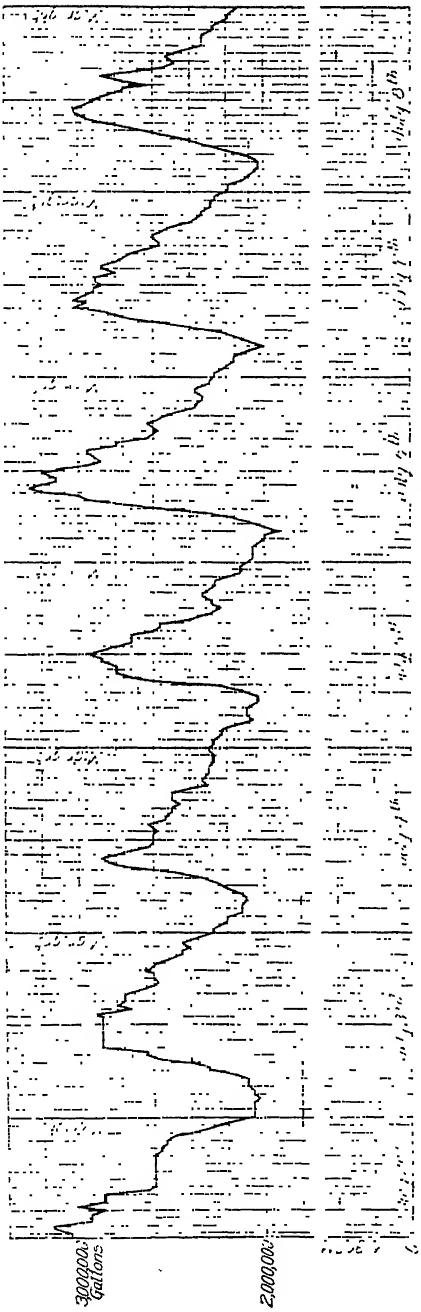


Fig. 43. Chart of Sewage Flow at Alliance, Ohio

about an oxidizing decomposition of the organic matter in sewage with but few or no objectionable odors; that *anaërobic* bacterial action, on the other hand, in the absence of free oxygen is usually accomplished with offensive odors mainly resulting from sulphurated hydrogen compounds which are formed thereby. The total number of bacteria in the sewage per cubic centimeter is indicative of the pollution, and serves as a comparison with the treated sewage as to degree of purification. Where disinfection of the sewage is desired, a count of the number of *b. coli communis* is made. This form of bacteria exists in the intestines of all animal life and is indicative of animal pollution. While it is in itself harmless, yet it is more easy to differentiate than the pathogenic bacteria, of typhoid, cholera, enteritis, and other water-borne intestinal bacteria which may or may not be present,

and as these bacteria are less hardy, the removal of *b. coli* is an indication of the removal of harmful bacteria.

DISPOSAL SYSTEMS

114. Requirements of Sewage Disposal. The requirements as to sewage disposal are varied for there are few cases where the conditions are the same. A study must be made of the conditions in each particular case, taking into consideration the location of water supplies; other industries which might be affected; the capacity and nature of the stream into which the sewage is to be discharged; the condition of the sewage itself; the possibilities of elimination of storm flows; and of the suitable location of treatment works. The variation of the hourly flow of the sewage must also be studied where treatment works are to be installed and data obtained on the amount of storm water admitted during rains.

Fig. 43 shows the weekly flow of the sewage at Alliance, Ohio. During the wet season this flow is tripled for brief periods.

There are many towns on large bodies of water where disposal by dilution is entirely feasible. On smaller streams, a partial purification in the nature of the elimination of solids or disinfection, or both, may be satisfactory, and in other cases, on small streams or in locations immediately above important water supplies, the highest degree of purification must be effected, including the removal of all organic matter and the disinfection of the bacterial content.

115. Classification of Methods. The methods employed in disposal of sewage are dilution and purification. The methods of purification now in use consist of broad irrigation, chemical precipitation, screens, settling tanks, septic tanks, contact beds, sprinkling filters, sand filters, disinfection, and electrolytic treatment. Each of these methods may be subdivided into various types, but in this book it is the intention in describing them, to outline only the most widely used and successful type under each. In considering the different methods, it is to be understood that one or more may be necessary to solve the problem, as will be outlined hereafter.

DILUTION

116. Controlling Factors. Nearly all of the larger cities of America dispose of their sewage by dilution. This is due to their location on the seaboard or large rivers.

For the proper dilution of sewage, there must be a volume of water sufficient to permit of *aërobic* bacterial action which will effect a complete breaking down of the organic matter and at the same time not destroy fish life; or in other words, the oxygen content of the stream must not be materially reduced. The minimum is set by authorities at from 30 per cent to 70 per cent of the saturation volume. The amount of water required to attain this result depends on the amount of dissolved oxygen in the stream and conditions for replacing it. Leading authorities estimate this at from 4 cubic feet to 10 cubic feet per second per 1000 tributary population. There must also be current sufficient to prevent silting-up of the stream or bay, and it is also important that there be no inshore currents which will deposit floating material on the shore lines. Large quantities of trade wastes from industries may kill fish life and must therefore be considered. The distance required to complete the purification by this method varies also with the character of the stream. A mountain stream with many waterfalls will manifestly purify itself much more quickly than a sluggish lowland river. There are no signs of pollution of the Mississippi at New Orleans and yet it receives above this point, the sewage of over 10,000,000 people. However, for the last 600 miles it travels through delta country where the drainage is away from the river.

117. Dilution of Chicago Sewage. The Chicago Drainage Canal is a notable example of disposal by dilution. This canal was constructed at a cost of $64,000,000 to divert the flow from the lake harbor through the Chicago River to the Illinois River, a tributary of the Mississippi. The sewage is diluted on a basis of 3.3 cubic feet per second per 1000 population, although the engineers in charge recommended a minimum of 4 cubic feet per second.

The Illinois River enters the Mississippi 357 miles from Chicago, a few miles above the St. Louis Water Works Intake. In a suit brought by the city of St. Louis to prohibit this arrangement, extensive tests showed that there was no substantial pollution from this source. Recent reports show, however, that for a distance of one hundred miles from Chicago along the Desplaines and Illinois rivers there is a very unwholesome condition and that along the Chicago River there are serious nuisances. Recommendations have been made to remove the solids before dilution.

118. Other Examples of Dilution. The sewage of New York is discharged through many outlets into New York Harbor. This has resulted in considerable deposits of silt at many points and studies are now being made of a partial purification of sewage from this metropolitan district.

The sewage from the main district of Boston is carried by an outfall sewer to an island in the outer harbor where it is stored in basins and discharged only during the early hours of the outflowing tide.

Many of the lake cities have long outlet sewers into the lake to prevent silting-up and deposit of solids on harbor front, and also to insure proper dilution.

BROAD IRRIGATION

119. General Principles. Broad irrigation is the oldest type of scientific purification of sewage. It has been practiced on a large scale in England and Germany, and several installations have been made in America. It consists in applying the sewage by a system of ditches to farm areas with the idea of irrigating them and also obtaining the fertilizing value of the sewage. The principle is that of *aërobic* bacterial action by natural filtration, and depends for success on a light and preferably sandy soil and on being able to operate uniformly at all times and seasons without overloading the treated area. An acre of area is the average requirement for each one hundred of tributary population. When this method first came into use, it was predicted that considerable profit would be derived. It has, however, been found that in the wet seasons it is very difficult to take care of the sewage and prevent water-logging the crops without by-passing the sewage; and that as a result the crops that can be raised are limited, and the fertilizing value of the sewage does not justify the expense required to apply it properly.

120. Efficiency of Broad Irrigation. The results obtained from well-operated irrigation farms are excellent. The effluent is stable with a marked reduction in bacterial count and in many instances showing absence of *b. coli.* Dr. Dunbar states that he is convinced that it would be cheaper for many towns to abandon irrigation and replace it with artificial biological processes and that

the day is not far off when Berlin will sell its irrigation farms for building purposes and construct artificial biological filters.

This is the attitude of all sanitary engineers at the present time. The only condition where it can now be favorably considered is in a district with low rainfall where irrigation is necessary for crop raising, and when the soil is adapted to irrigation.

CHEMICAL PRECIPITATION

121. Controlling Factors. The method of sewage disposal by chemical precipitation was introduced by various private companies under patents at about the same time that broad irrigation came into use, and there have been some large installations in England, Germany, and America.

The sewage is introduced into settling tanks where it is treated with chemicals such as sulphate of iron and lime. These chemicals form a heavy flocculent precipitate which settles in the tanks and carries with it a part of the suspended matter in the sewage. The precipitated material, or sludge, is then drawn off and usually compressed by sludge presses so as to remove the water and facilitate handling.

122. Efficiency of Chemical Precipitation. It has been found that 90 per cent of the total suspended matter and bacteria can be removed from sewage by this process. The effluent is putrescible as there has been no change in the remaining organic matter.

When this process was first installed, fabulous claims were made of the value of the sludge as fertilizers from plants of this type. This has been much overrated and it is difficult to get farmers to come to the plants and haul it free. Many such plants have to deposit the sludge in fills and as the amount will average 5 cubic yards per million gallons of sewage treated, there is a considerable quantity for a town of any size. The high cost of chemicals and labor required in the operation has also been against this method which is no longer being installed for municipal disposal works.

123. Conditions Favoring Chemical Precipitation. There are some circumstances in dealing with trade wastes or some special conditions, such as at London, under which the chemical process can be used to advantage.

It has been necessary at London to remove the suspended matter to prevent the silting up of the Thames. This has been accomplished by treating the sewage by chemical precipitation amounting to 200,000,000 gallons daily in nineteen settling tanks having a combined capacity of 44,000,000 gallons. About 8,000 cubic yards of sludge is deposited daily. This is pumped into tank steamers and carried out to sea.

SCREENS

124. Purpose. Coarse screens are in general use at sewage pumping stations and disposal works to remove the coarse suspended matters. During the last few years, mechanically operated fine screens have been developed in Germany which can remove as high as 80 per cent of the suspended matter from the sewage. These fine screens are now being introduced into America.

125. Coarse Screens. The most common type of coarse screen is the bar screen consisting of vertical steel bars spaced from ½ inch to 1½ inches apart depending on conditions, and arranged in a masonry pit at the outlet end of the sewer across the line of flow of the sewage. The fibrous materials in the sewage make the problem of cleaning a difficult one and screens must, therefore, be installed in duplicate so that one can readily be removed. For small plants, the screens are usually cleaned by the attendant pulling a garden rake up over the vertical bars several times daily. In large plants, one screen is removed from the pit by hoists or hydraulic lift and cleaned with a hose on the operating floor above, or some form of mechanical cleaning device is used.

The material obtained from coarse screens consists of rags, paper, sticks, lemon peels, and other coarse organic matter. This is usually placed in large cans and hauled to a dumping ground where it is buried. If the city owns an incinerating plant, it can be mixed with the garbage and burned.

126. Fine Screens. There have been many types of fine screens developed, with varying success. The best known type at the present time is the Reinsch-Wurl screen as shown in Fig. 44. These screens consist of a large circular plate which is placed at an inclined angle in the outfall sewer and is revolved about its center. This plate is perforated over its entire surface with fine

slots. The size of slot and diameter of plate vary with the amount of sewage to be treated and the degree of purification to be effected.

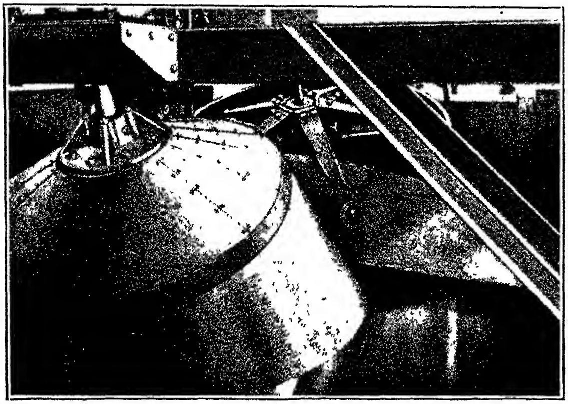


Fig. 44. Reinsch-Wurl Screen in Experimental Plant, Dresden, Germany

As the plate revolves, the deposited solids are brought above the surface of the sewage, Fig. 45, and are then brushed off the plate

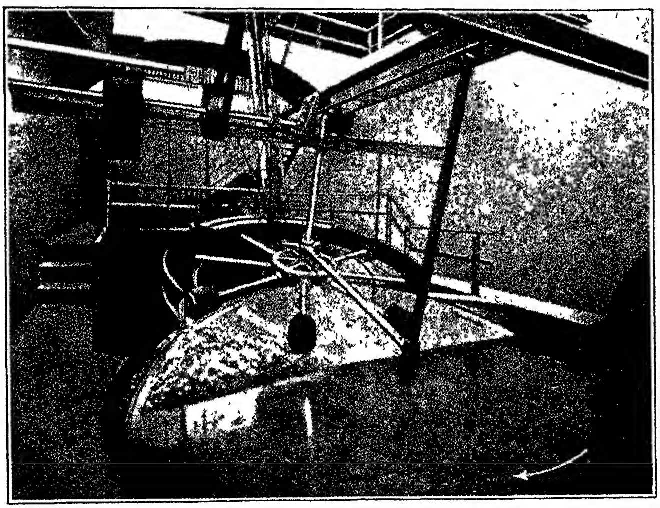


Fig. 45. Reinsch-Wurl Screen in Operation at Bremen, Showing Brushes

by metallic brushes which sweep the screenings into a trough where it is transported to the sludge presses or carted away.

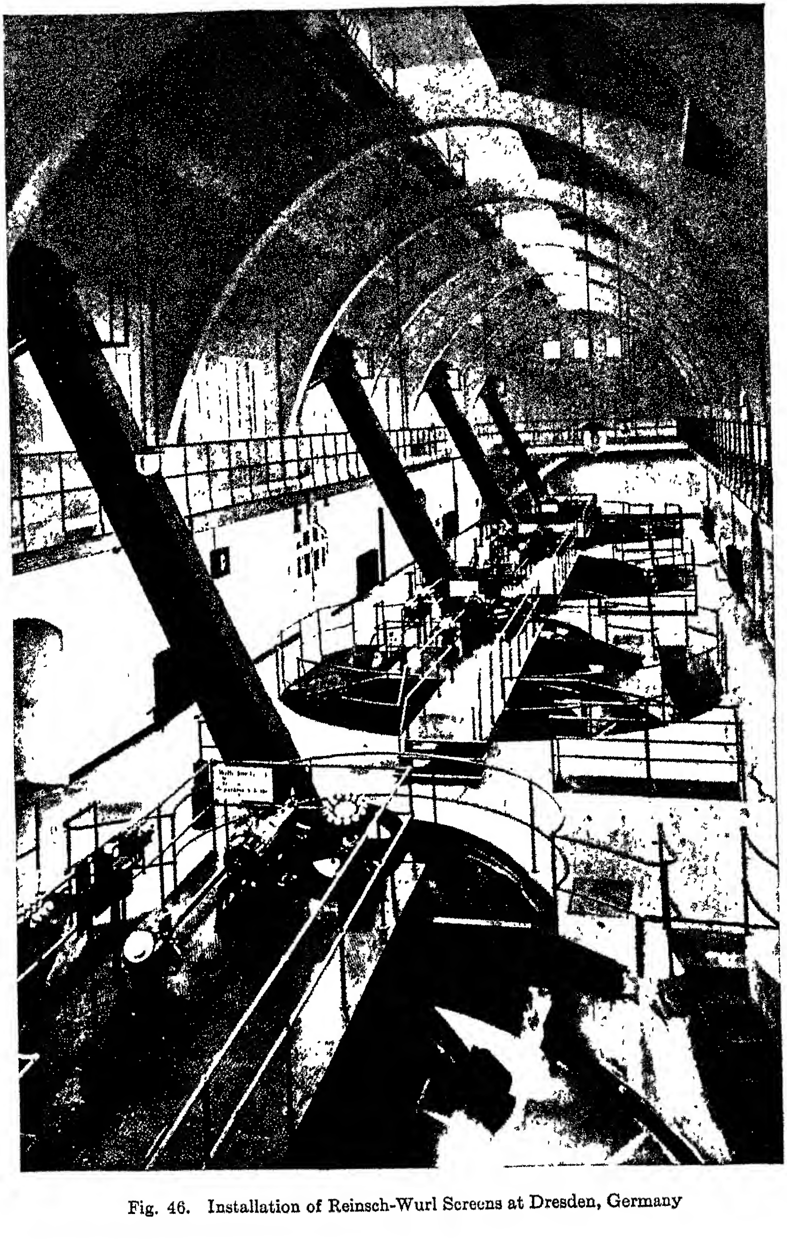


Fig. 46. Installation of Reinsch-Wurl Screens at Dresden, Germany

The amount of power required to drive this apparatus is small and the installation is much less expensive than settling tanks. There are the difficulties, however, of a higher operating cost and the problem of the distribution of the screenings. The screenings can be disposed of as outlined under the paragraphs on coarse screens, but on account of their amount, it is preferable to make some arrangement regarding their use as fertilizers with the farmers, who will take them free unless there is local prejudice. While these screenings have some value, yet they must be removed daily and disposed of immediately before putrefaction sets in, and farmers will not agree to an annual contract on better conditions than free material, on account of the difficulty of distributing it in winter weather.

The plant at Dresden, as shown in Fig. 46, is the largest plant of this type in the world. It was installed in 1911, and treats a maximum flow of 4500 gallons per second. For three months in 1911, the Elbe, into which the effluent is discharged, had a flow of less than one-half that of the incoming sewage and yet no nuisance was caused thereby.

These fine screen installations are specially adapted to use where clarification alone is required for conditions under which it is necessary to install the purification works adjacent to a built-up district. The fresh screenings have not had time to putrefy and can be hauled away easily without offense. They are also used for clarification and the recovering of by-products of industrial waste, and under special conditions for screening sewage to be treated by filters.

SEDIMENTATION TANKS

127. Efficiency. The method generally used for clarifying sewage is by natural sedimentation. This will remove from 50 per cent to 75 per cent of the total suspended matter from the sewage and 35 per cent of the organic matter, leaving the effluent with the balance of the organic matter in solution and in suspension as colloids. Sedimentation is used as a method of clarifying sewage before it discharges into a stream where dilution is feasible, or as the first step in complete purification.

128. Classes. There are three general classes of sedimentation tanks: (1) grit chambers; (2) settling tanks; and (3) septic tanks. These classes have various modifications and designs.

129. Velocity. Practically all designs are based on a continuous flow through the tanks at a velocity sufficiently low to deposit the suspended matter. Extensive experiments have been made to determine the carrying velocity of sewage-laden water, and as a result of these experiments authorities place the maximum velocity for settling tanks at one-half inch per second, and the average velocity of grit chambers at 1 foot per second.

Grit Chambers

130. Use. Grit chambers are used on combined sewer systems or sanitary systems which admit some street drainage, in order to remove the coarse organic suspended matter before it has reached the pumps or disposal works. Where the sewer system carries only house drainage, they are unnecessary.

131. Design. They must be designed so that no organic matter will be deposited and so that the period of retention is not long enough to start septic action. A velocity of 1 foot per second will deposit the grit without the organic matter and the period of retention may vary from a few seconds to 5 or 10 minutes, depending on the coarseness of the material. Where the amount of sewage varies at different times, several compartments must be installed with automatic overflow weirs so that the velocity and period of retention may be uniform. The usual design is rectangular in plan with length sufficient to give the required period of retention and velocity, and with a cross-section suited to securing a uniform velocity over the entire area without complicated baffling. The bottom should be sloped on a 10 per cent grade to an outlet drain controlled by a valve. The details of design are very similar to those for the more complicated settling tanks, which will be hereafter described. The depth used, however, is comparatively shallow, as grit chambers must be cleaned frequently, and too much surplus storage capacity would affect the uniform velocity desired.

Settling Tanks

132. Settling vs. Septic Tanks. Settling tanks can be distinguished from septic tanks by the fact that putrefactive action in the latter results from the action of *anaërobic* bacteria on the organic matter retained therein. An arbitrary definition of septic

tanks has been that they must have an uninterrupted flow without removal of sludge for at least six weeks. However, newly cleaned settling tanks have shown signs of septic action in a few days after being placed in commission. Settling tanks retaining their solids are, however, usually distinguished from septic tanks by the frequent cleaning periods in the former case, and the six weeks' period is usually taken as the dividing line.

133. Basic Conditions. Settling tanks may be divided into two classes, single-story tanks and two-story tanks. The governing criteria for both classes are a maximum velocity of one-half inch per second, a minimum retention period of one hour, and a minimum distance of horizontal travel of 35 feet.

134. Single-Story Settling Tanks. *Design.* The usual type of the single-story settling tank is rectangular in plan and has a continuous horizontal flow lengthwise through the tank. Several compartments are constructed to permit one or more tanks to be used in proportion to the flow of sewage and to permit tanks to be cleaned without interfering with the continuous operation of the plant. The depth of tanks should preferably be 12 feet to 16 feet to give ample room for storage of sludge without its being disturbed by the flow of sewage. Their capacity should be a retention period of from 1 hour to 4 hours with additional time sufficient for sludge storage. Their length and width are governed by the number of units, the capacity and limitation of velocity and horizontal travel given above. The length does not usually exceed 100 feet and the width is usually $\frac{1}{8}$ to $\frac{1}{10}$ the length. Covers over tanks are not necessary, although they are usually installed on well-designed tanks for the sake of appearance. If covers are used, vents must be installed for gases. It is important to obtain a uniform distribution of the sewage across the entire cross-section and to maintain a uniform velocity. This is accomplished by distribution across inlet and outlet ends and by one or more baffles across the tanks. It is very necessary to design the tanks so that they can be cleaned easily. The bottoms must be sloped on a minimum slope of five per cent to a central sump where the sludge can be drained by gravity to a sludge bed or can be pumped. A fire hose with water under good pressure is indispensable in economic cleaning. The sludge bed must be constructed of gravel or other porous

material and be well underdrained. The minimum depth must be at least 12 inches, and the surface must be covered with a minimum depth of 2 inches of sand to retain the sludge. The sludge bed must be of area sufficient to permit the sludge to be deposited with a maximum depth of six inches so that it can dry. With this depth it will dry under favorable conditions in from one to two weeks. With well-digested sludge, an area of 0.3 square feet per inhabitant is sufficient.

135. Sludge. The sludge problem is the most serious one to be considered with settling tanks. If they are frequently cleaned, this highly putrescible organic matter must be discharged onto the sludge beds every few weeks, causing very disagreeable odors; and when the sludge is scraped off after drying, it must be disposed of. Where the number of tanks is sufficient to permit, one should be placed out of commission so that the sludge can digest in the tank before being discharged, as a period of several months is required to obtain thorough digestion. This is accomplished by the action of *anaërobic* bacteria, which usually produce offensive gases and cause a highly septic liquid in the tank above the sludge. It is therefore dangerous to install plants of this type within half a mile of residences.

Septic Tanks

136. Basic Conditions. These tanks are the same as the single-story settling tanks previously described, except that septic action or the action of *anaërobic* bacteria is desired. The tanks are designed with a capacity of from four hours' to twenty-four hours' settling period so as to insure capacity enough for sludge storage. In America the maximum is twelve hours. The flow through the tanks is continuous and at a slow velocity, exactly as described for settling tanks.

As the sewage flows slowly through the tank, the coarser suspended matter settles to the bottom where it is attacked by millions of *anaërobes* which have developed on the sludge previously retained. Lighter particles rise to the surface forming a thick heavy scum over the entire surface. Small atoms breaking off from this sink to the bottom, while light ones rise from below, impelled by gases. All these are teeming with *anaërobes* which are liquefying and gasi-

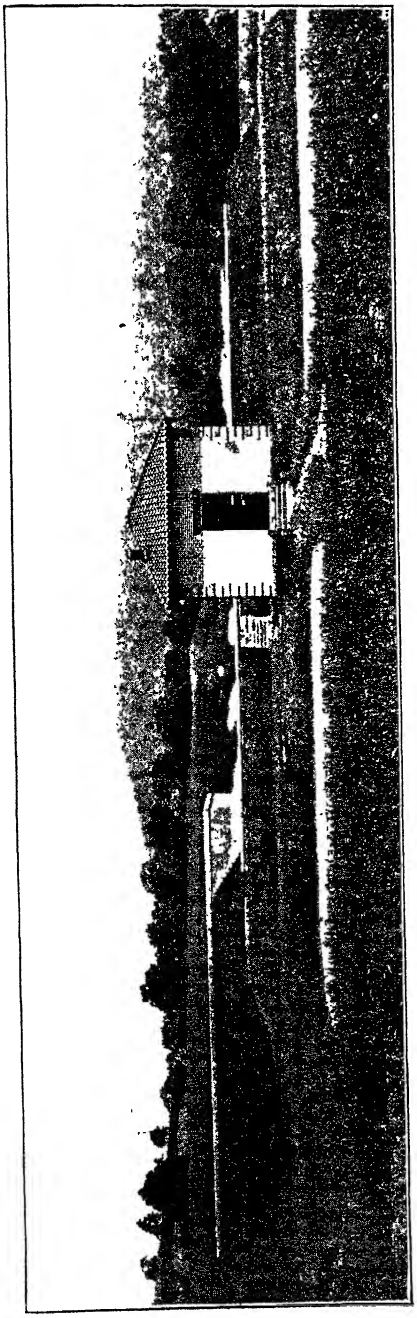


Fig. 47. General View of Sewage Disposal Plant at Polk, Pennsylvania

fying the organic matter. In septic tanks sludge is only removed when there has been an accumulation such that it overtaxes the sludge capacity. Where *anaërobic* action works perfectly, the greater proportion of the suspended matter is liquefied, and it has been found that after retention for three or four months the remaining sludge is inodorous, while in many cases it is removed once a year with little or no nuisance. The difficulties experienced with septic tanks are from the offensive gases usually present in the tank effluent. These gases are liberated when the effluent is discharged onto the biological filters, with resulting nuisance to adjacent property. The oxygen content of the sewage is also removed, which places it in bad shape for *aërobic* treatment either by dilution or filtration.

Where septic tanks work perfectly with no attendant odors in the effluent, they furnish one of the best methods for removing the suspended matters. Most of the single-story settling tanks in America are

operated as modified septic tanks. The storage capacity of the sludge is in many cases limited, requiring frequent removal of same, but there is a considerable liquefaction and the sludge fairly stable. It is difficult, however, to prevent fresh sludge that has just been deposited from being drawn off with the more thoroughly digested material under these conditions.

137. Polk Tanks. Figs. 47 and 48 show an installation of settling tanks of this type which are used to remove the suspended matter before the sewage is applied to sprinkling filters. Figs. 49 and 50 show the details of these tanks.

This plant was designed to purify the sewage from one of the largest state institutions in Pennsylvania, and to discharge it into

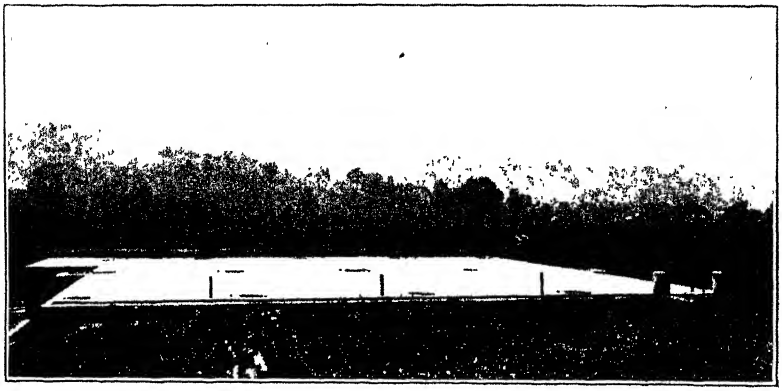


Fig. 48. Covered Settling Tanks of Polk Disposal Plant

a small stream of practically pure water where it was impossible to obtain a flow in the dry season sufficient to render the effluent from the sewers inoffensive or properly diluted. The plant was designed to treat daily five hundred and sixty-four thousand gallons. It consists of screen chambers, settling tanks, sprinkling filters, and disinfection apparatus, for the disinfection of the effluent with chloride of lime.

There are four settling tanks, Fig. 48, each 80 feet long by 16 feet wide by 10 feet deep, and each with a capacity of 96,000 gallons, which permits of a settling period of 12 hours with three tanks in operation.

As will be noted in Figs. 49 and 50, the sewage is admitted into a reinforced-concrete distributing trough extending across

the inlet ends of the entire group of tanks, from which sewage can be admitted into one or more compartments through two or three

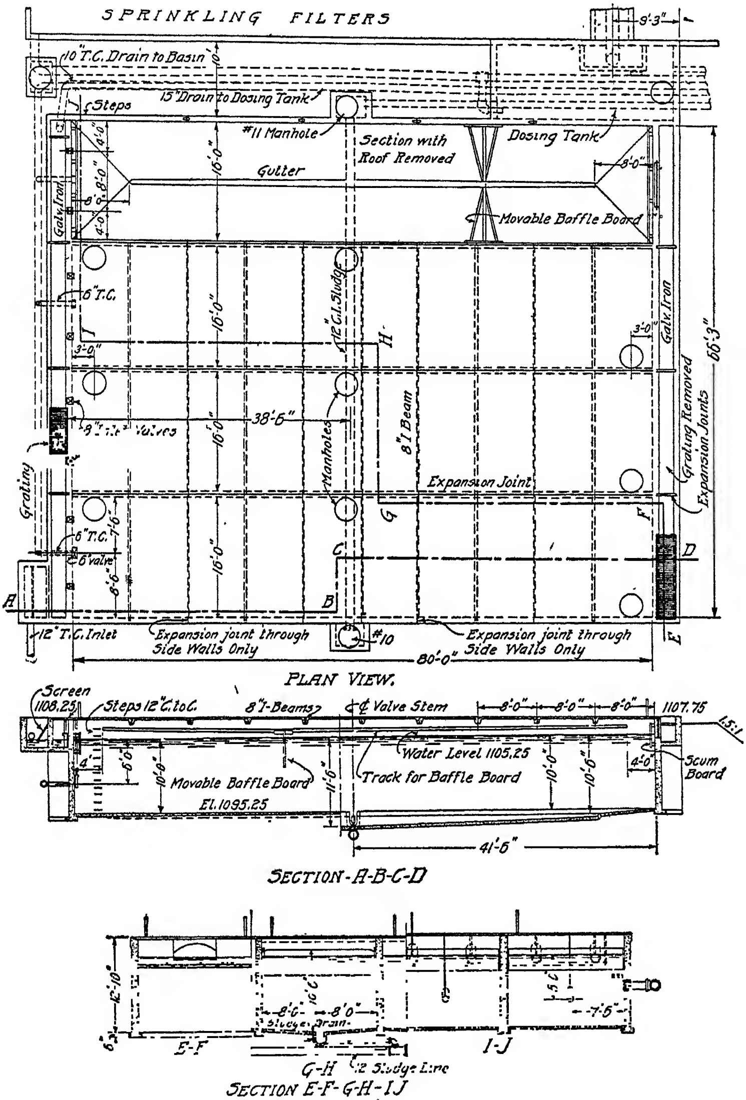


Fig. 49. Detailed Plan and Sections of Covered Settling Tanks

gate valves located immediately below the flow line. A distributing baffle of wood is constructed across the inlet end of each tank oppo-

site these gate valves at a distance of 10 inches from the wall and extending to a depth of 2 feet below the flow line. This baffle distributes the sewage uniformly across the inlet end of the tank. At the outlet end of each tank the sewage is removed over a steel weir 6 feet long located at the flow line of each compartment and at the center of the wall. This weir is also protected by a wooden

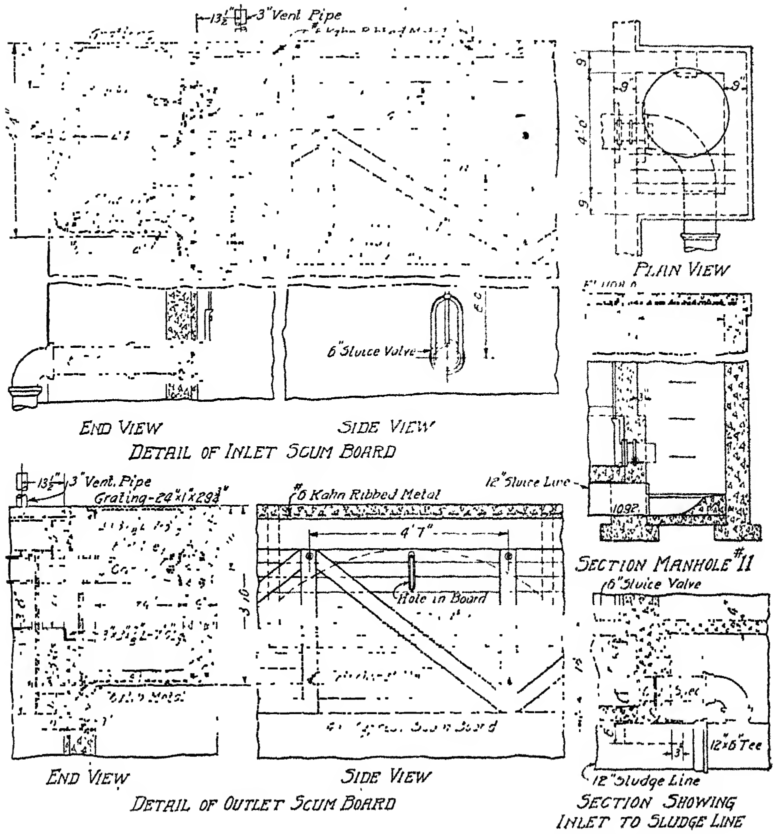


Fig. 50. Diagram Showing Special Details of Polk Covered Settling Tanks

baffle to assist in taking off the sewage uniformly from the entire width of the tank and to protect the outlet from floating material or from solid matter that may be working up from the bottom of the tank because of septic action.

A movable wooden baffle extending 4 feet 6 inches below the flow line is suspended on a trolley running the length of the tank

and can be located at any point between the inlet and outlet ends as may be desired. This baffle is also used to prevent cross-currents and to assist in a uniform flow through the tank. It will be noted by further reference to the illustrations that stop planks are arranged on the outlets of the various compartments and on the inlet and outlet troughs opposite the partition walls between the tanks. By adjusting these stop planks the tanks can be operated in series instead of in parallel.

It will be noted that, for the purpose of cleaning, the concrete bottom of each compartment slopes on a 5 per cent grade to a gutter extending lengthwise through the center of each tank, as shown in section *GH* in Fig. 49. The bottom of this gutter is also placed on a 5 per cent slope and at the center of the tank there is a 6-inch valve connection for draining off the sludge to a 12-inch sludge line, extending under the tanks and carrying the sludge by gravity to sludge beds. It will also be noted that the tanks are covered with concrete roofing and that the inlet and outlet troughs are covered with cast-iron gratings which serve to improve the general appearance.

This plant has been in operation for over five years, and the results obtained from these settling tanks have been highly satisfactory. They have maintained uniformly over fifty per cent removal of the suspended matter and there has been a very small accumulation of sludge, most of it being liquefied in the tanks. A small amount is removed at frequent intervals and discharged on the sludge bed and when dried is scraped off and plowed into adjacent ground. There have been no complaints of offensive odors and practically no trouble from gases liberated by *anaërobic* action. The plant is, however, well isolated, being three thousand feet from the institution. It is typical of the higher grade settling tanks in America and if it were not for the frequent removal of sludge, thereby preventing complete septic action, it would be typical of the septic tanks.

138. Two-Story Tanks. *Basic Conditions.* The problem of eliminating the nuisance arising from the use of settling or septic tanks where offensive odors are in many cases given off by the effluents due to the *anaërobic* action in the compartments, and the difficulties experienced in handling the sludge and obtaining thorough

digestion of it before it is removed from the tanks, has resulted in the development of a type of tank where the sewage flowing through the tank is kept entirely separate from the deposited suspended matters. It has been found also that septic action does not benefit the liquids for the secondary treatment on the biological filters, but that in fact it is preferable to get the liquids to the biological filters in a condition as fresh as possible, in order to retain some oxygen which would be favorable in the secondary treatment. The typical tank of this type is the Imhoff tank developed by Dr. Karl Imhoff, the German expert.

Design. In this type of tank the sewage flows through an upper compartment under the conditions previously specified

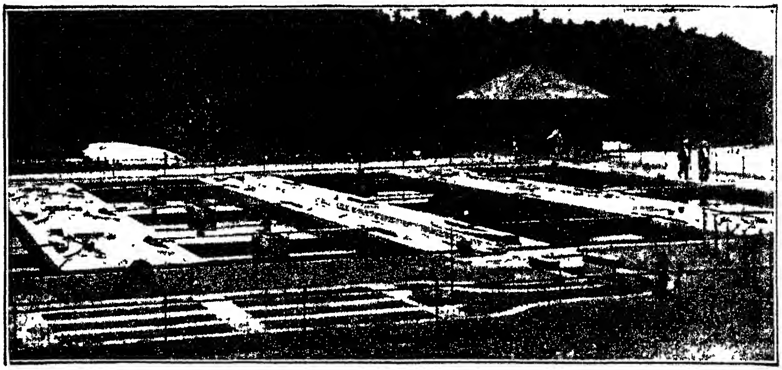


Fig. 51. Installation of Imhoff Tanks at Atlanta, Georgia

for travel and velocity, but with a retention period of from one to four hours. The compartment is equipped with a sloping bottom placed on a slope of not less than 1.2 vertical to 1 horizontal, and terminating in a sealed slot, with a lap horizontally of at least 8 inches. This slot discharges into a lower compartment that has no connection with the upper compartment other than the sealed slot. This lower compartment is designed to retain a capacity of at least six months' sludge, based on a capacity of 1000 cubic feet per thousand inhabitants. The lower compartment is usually built cone shape, terminating in a sump at the bottom from which there is a discharge line for the sludge bed. There must also be an opening from the compartment to the surface as a discharge for gases and as an admission for cleaning. As the sludge is drawn

from the bottom of the compartment, and as there is six months' capacity for *anaërobic* action, if the sludge is drawn off in small

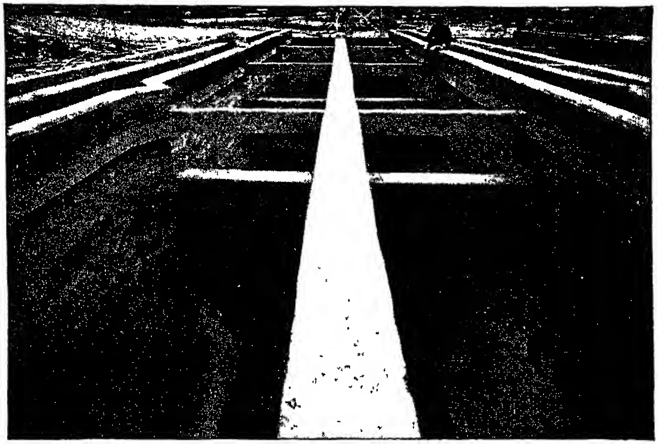


Fig. 52. Interior View of Imhoff Tanks at Batavia, New York

Fig. 53. Installation of Imhoff Tanks at Essen, North Germany

quantities at intervals of six weeks, only thoroughly worked over sludge will be placed on the sludge beds and the result is that there can be no nuisance.

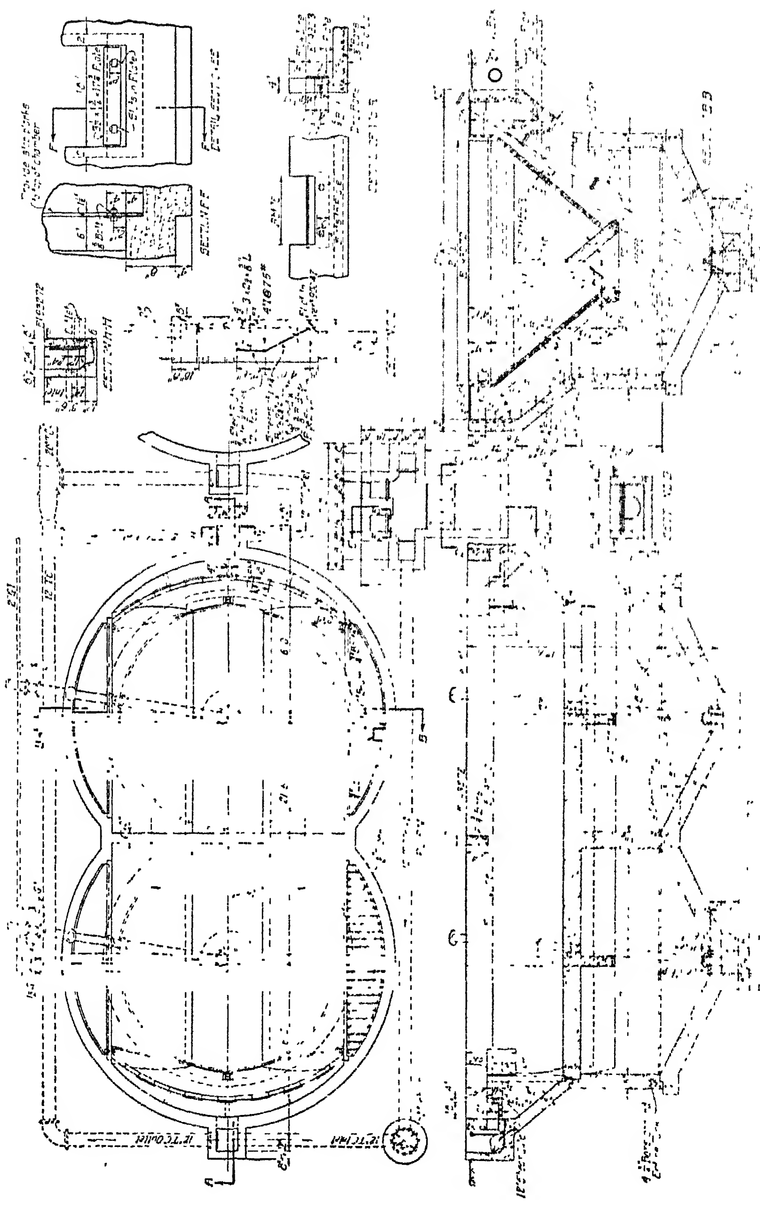


Fig. 51. Plan and Sections of Imhoff Tanks at Greenville, Pennsylvania

Operating Results. Figs. 51 to 53 show installations of this type of tank at Atlanta, Georgia, Fig. 51; at Batavia, New York, Fig. 52; and at Essen, North Germany, Fig. 53. The Atlanta plant was placed in service in 1912, and the result of the operations for the years 1913 and 1914, which was recently published, showed an average removal of total suspended matter by the Imhoff tanks of 80 per cent. The average period of retention in the flow of sewage was two hours. The plant at Batavia, New York, which was installed in 1912, is also giving excellent results. No nuisance has been noticed at either plant and the sludge removed from the tanks has been well-digested and inodorous.

139. Greenville Tanks. There are various forms of design for the Imhoff tanks. Fig. 54 shows the plans for the Imhoff tanks to be installed for Greenville, Pennsylvania. These tanks are of the circular type, constructed in pairs and arranged for a longitudinal flow through each pair. Piping facilities provide for reversing this flow, a necessary operation every few weeks in order that the deposited material may be uniformly distributed in both tanks. The entire capacity of this plant is one million gallons per 24 hours based on a settling period of $1\frac{1}{2}$ hours, and on a sludge storage of 6 months for seven thousand people. Imhoff tanks are constructed of reinforced concrete and the arrangement for admitting sewage to the tanks and reversing its flow is by means of cast-iron pipe lines controlled by concrete manholes with stop planks as shown. Sewage is distributed across the inlet end of the tank by concrete weirs spaced uniformly along the outer edge of a trough of the same material built across the entire width of the tank at the inlet end and protected by a wooden baffle extending to a depth of 24 inches below the flow line. This baffle serves to distribute sewage uniformly across the entire tank. Sewage is taken off at the opposite end of the pair of tanks by a concrete trough similar to the one described, also protected by a skimming baffle. The upper compartment is separated from the lower compartment by a reinforced-concrete slab, terminating in two slots on each side of a wedge frame built at the bottom of the compartment as shown. (See section *BB* of Fig. 54.) It will be noted in the design, that the upper or settling compartment is common to the two tanks, but that the lower or sludge compartments are entirely independent

of each other. The sludge is removed from a sump at the center of the sludge compartment of each tank by means of an 8-inch sludge pipe extending to within a few feet of the flow line of the tank, where it is connected by a valve to a gravity drain extending

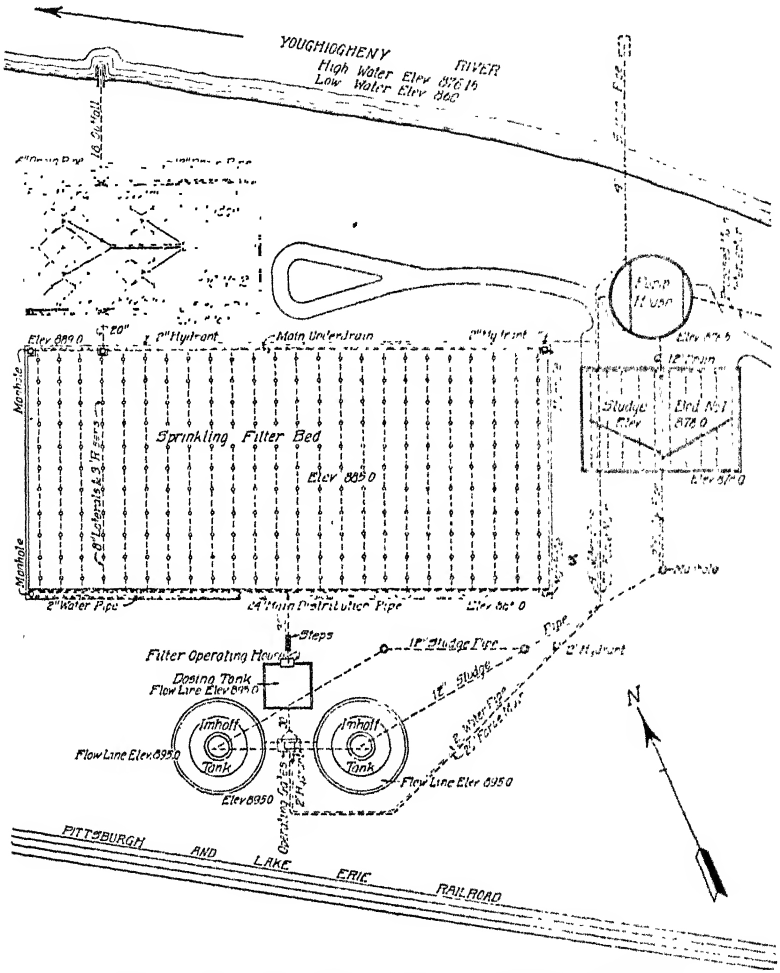


Fig. 55. Plan of Sewage Disposal Plant at Connellsville, Pennsylvania

to the sludge bed. Without interfering with the operation of the tank, sludge is removed by hydraulic pressure in the tank upon opening this valve. It will be further noted by reference to the illustration that ample area is left along the sides of the settling

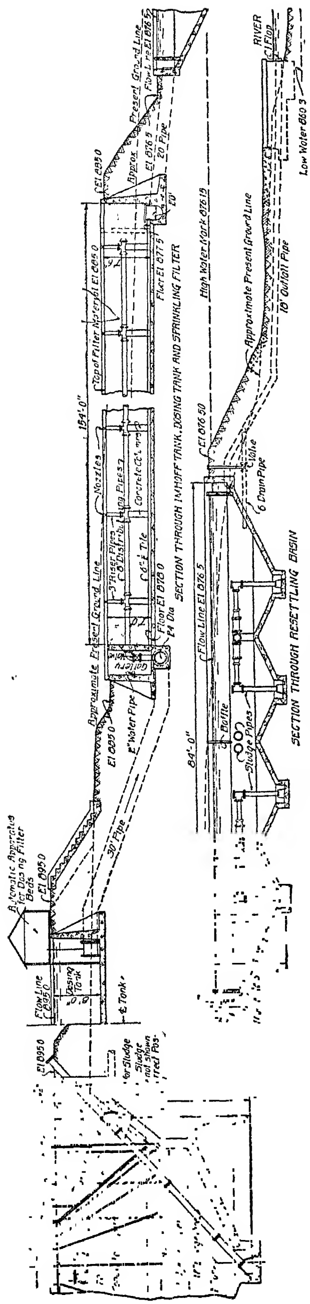


Fig. 56. Section of Sewage Disposal Plant at Connellsville, Pennsylvania

compartment at the top of the Imhoff tanks for ventilating the lower compartment. These compartments are covered with removable wooden gratings as shown.

140. Radial-Flow Tanks. Figs. 55 and 56 show a typical design of what is known as the radial-flow Imhoff tanks. These tanks were designed for a sewage disposal plant for Connellsville, Pennsylvania, a town having a population of sixteen thousand and a flow of sewage of three million gallons per 24 hours. Two radial-flow Imhoff tanks are planned for this installation. They are designed for two hours' capacity in the settling compartment and for a six months' storage of sludge, and are to be constructed of reinforced concrete. Sewage will be admitted through a manhole between the tanks, Fig. 55, from which it will be diverted to one or both of the tanks by means of stop planks controlling 20-inch cast-iron pipes extending to the circular trough near the center of each tank. This circular trough will be 14 feet in diameter and 27 inches wide, and will have slots in the bottom of it spaced uniformly around the circumference at a depth of 3 feet below the flow line. The diameter of each tank will be 58 feet. The settling chamber will be separated from the sludge chamber by a cone diaphragm of concrete as

shown, with a slot located at the bottom of the diaphragm, between it and the outside sloping wall of the tank. Halfway across the settling chamber between the circular inlet and outlet troughs, there will be a concrete baffle extending to a depth of half the tank at this point from the top. The outlet trough will be arranged with outlet weirs spaced along the top of the trough at uniform distances and at the same level. The sludge compartment will be drained by a sludge pipe extending from a sump at the center of the compartment to a gate valve located outside of the tank and 5 feet below the flow line. By opening this valve the sludge will be drained by hydraulic pressure without interfering with the operation of the tank.

Flushing. One of the most important adjuncts to equipping Imhoff tanks is a good fire hose with plenty of water pressure, and with facilities for applying it to any portion of the tanks. This is valuable in breaking up or removing any scum formation in the settling compartment, in keeping the sludge pipe clean, and in washing off any compartment which may be drained.

CONTACT BEDS

141. Use. Contact beds are a type of the biological filters generally used for treating sewage from which most of the suspended matter has been removed by sedimentation or by fine screens. They are employed to further remove additional organic and suspended matter and to render the effluent non-putrescible.

142. Basic Conditions. Contact filters are constructed of broken stone, hard slag, or well-burned cinders, preferably of material ranging in size from $\frac{1}{4}$ inch to 2 inches. The filters are usually of an effective depth of from 1 foot to 5 feet, depending on the amount of head available. The best depth is 4 feet to 5 feet and with this depth a maximum flow of 700,000 gallons of clarified sewage can be treated per acre. As a general rule 150,000 gallons per acre is the maximum amount that can be treated for each foot of filter depth. The beds must be operated so that they will be frequently filled with air, as the action is entirely that of *aërobic* bacteria. These bacteria exist in enormous numbers over the surface of the filtering material throughout its entire depth. When the sewage is placed in contact with this filtering material, the remain-

ing suspended matter in the sewage consisting mainly of colloids and non-settling solids, is to a great extent retained in contact with the filtering material by attrition; and the enormous bacterial growths of *aërobic* bacteria attack this organic matter, quickly reducing it to inorganic forms.

143. Design. Most of the contact filters installed in America are operated on the fill-and-draw method, being controlled by automatic apparatus which admits the sewage to one unit of a group of filters and when this is full opens up the outlet from it, and at the same time starts the sewage flowing into the next filter. These filters must be constructed as water-tight compartments and are usually built of concrete. The number of groups to be used and the size of units in each group depend upon the amount of sewage to be handled and the depth of filter desired. It is usually not desirable to have the units larger than a quarter of an acre each, and, on the other hand, it is advisable to have at least four units in order to secure long resting periods between the dosing. It has been found that the period of retention has very little to do with the efficiency, so that in most installations, the apparatus is arranged in such a way that the tank starts to empty a few minutes after it has filled.

144. Alliance Filters. Figs. 57, 58, and 59 show a plan of the plant and the contact filters of Alliance, Ohio, and the automatic control apparatus for one group.

As will be noted upon reference to the plans, the Alliance filters are designed to treat sewage clarified by settling tanks. They have a capacity for two million gallons of sewage per day. They consist of three groups, each with a total area of one acre and subdivided into four filters. Each filter has an effective depth of 5 feet and consists of a concrete compartment filled with well-burned cinders and underlaid by tile drains upon the floor, which drain to the central control chamber. The sewage flows from the settling tanks to the central control chamber of each group of contact beds, where it is distributed by automatic air-lock apparatus on to each bed at the surface. This apparatus consists of Miller-Adams siphons, each of which is controlled by an air bell which can break the siphon seals. These air bells are located in concrete compartments and connected by small pipes to the siphons. As the water

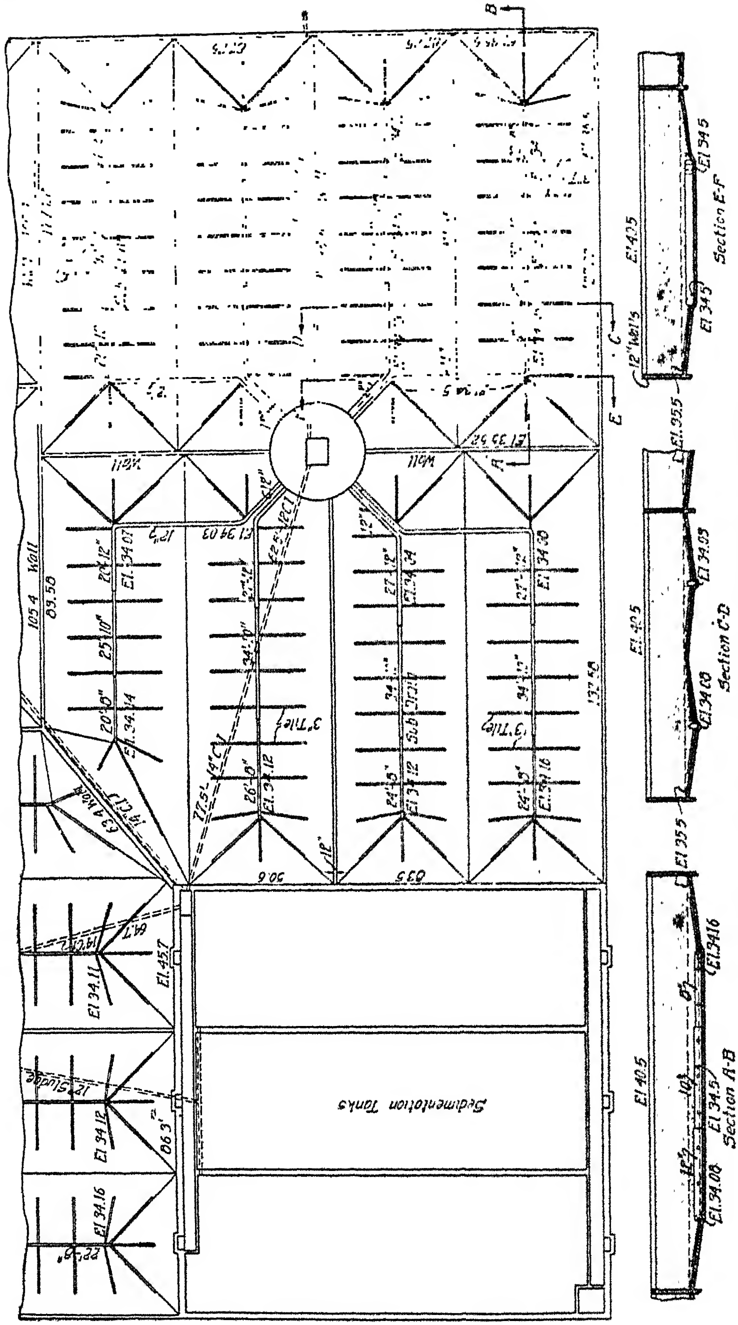


Fig. 57. Plan and Sections of the Contact Beds at Alliance, Ohio

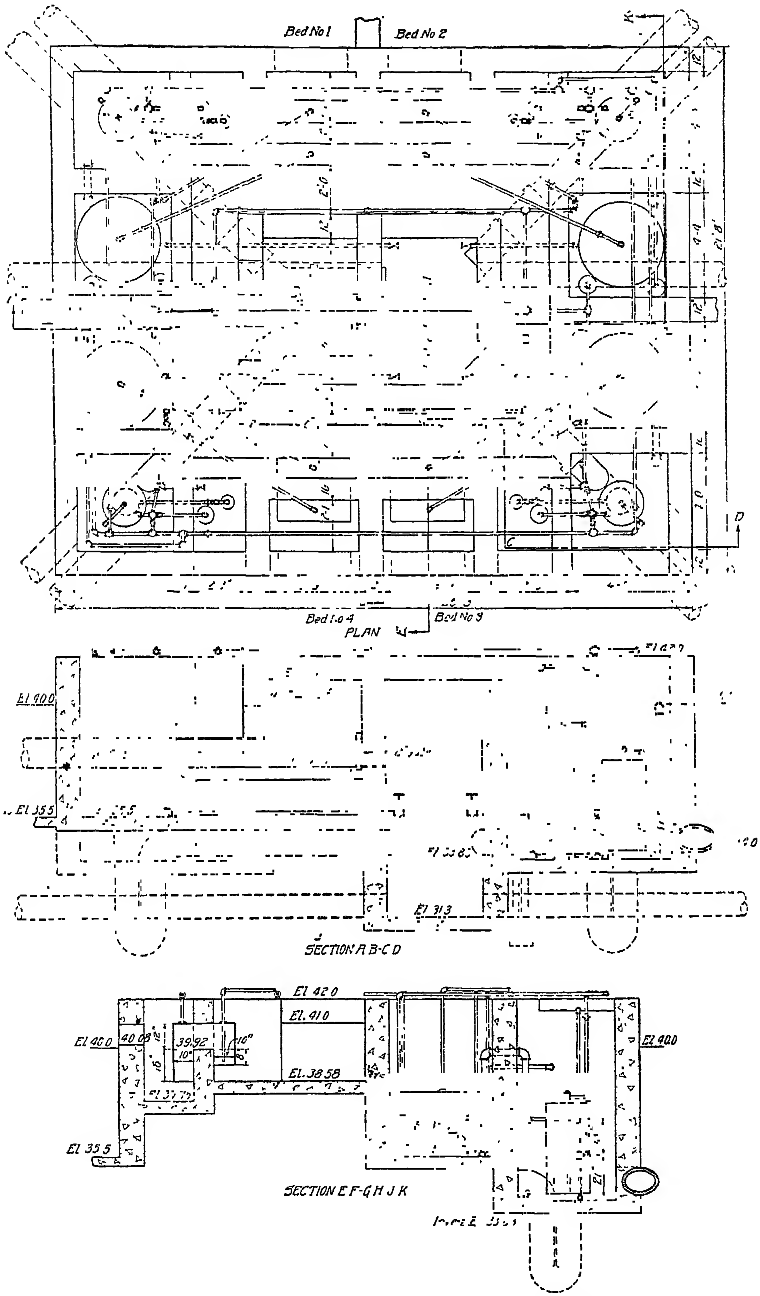


Fig. 58. Plan and Sections of Automatic Control Chamber for Contact Beds at Alliance, Ohio

rises in these compartments which are connected up to the filters, it displaces the air in the bells, gradually compressing it until it is of pressure sufficient to displace the sewage in the connected siphon, so that when one filter is full, it closes itself, opens up the inlet valve to another filter, and then opens up the outlet valve of the filter just filled.

145. Head Required. Contact filters are well adapted to gravity filtration plants where the loss of head due to the operation of the plant is limited to 6 feet or 8 feet. They are much more expensive in construction than sprinkling filters, which are less

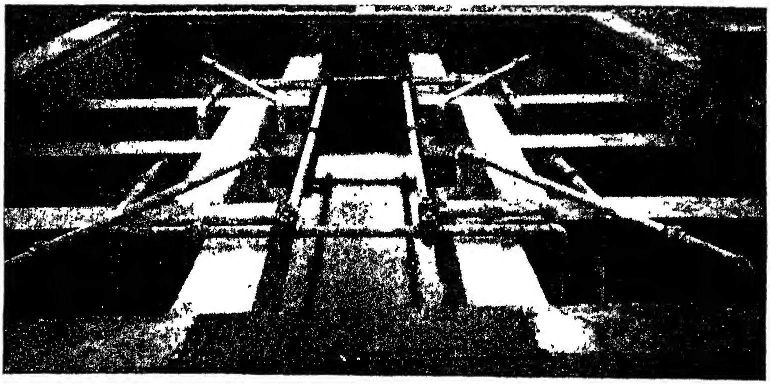


Fig. 59. Automatic Control Chamber for Contact Beds at Alliance, Ohio

likely to clog up and which give equally good results. Sprinkling filters, however, require at least eleven feet of head.

SPRINKLING FILTERS

146. Use. Sprinkling filters operate on the same principle as contact filters and are used for the same purpose, although the method of application of the clarified sewage is entirely different. They are essentially biological filters depending upon the action of *aërobic* bacteria, and the same method of removing the remaining suspended and organic matter from the sewage is carried out in the sprinkling filters.

147. Design. Sprinkling filters do not require water-tight concrete compartments and do not have to be subdivided. They can, therefore, be constructed much more cheaply than contact beds. They are built with an effective depth of from 5 feet to 8

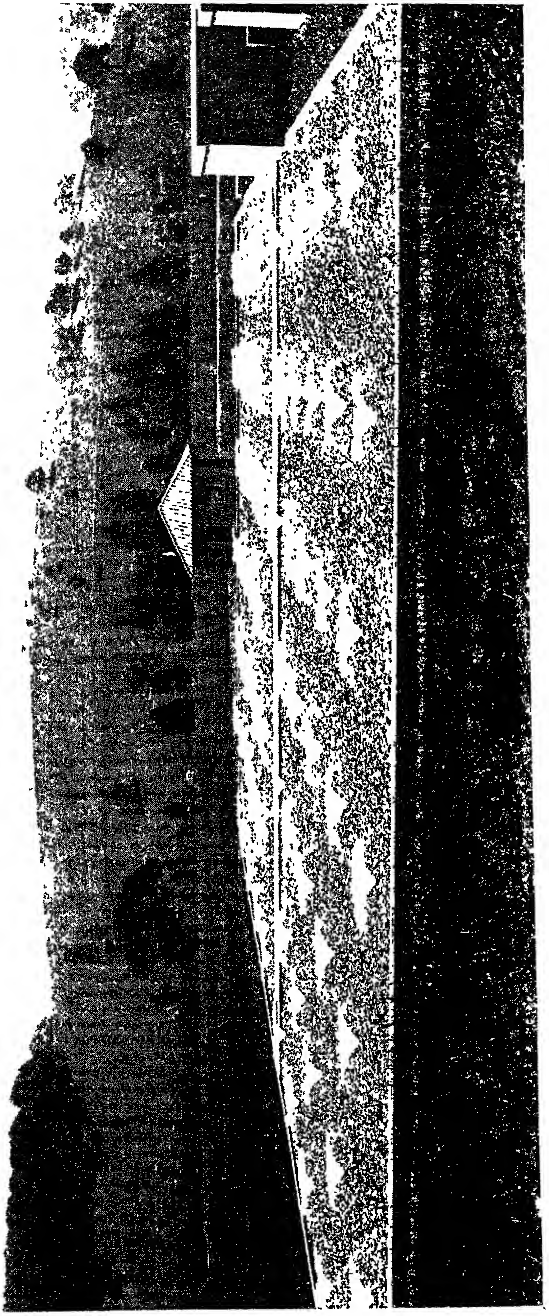


Fig. 60. View of Sprinkling Filters in Operation at Polk, Pennsylvania, Disposal Plant

feet and are constructed of broken stone or other material with a dense clear fractured surface of a size ranging from 1 inch to 3 inches. The sewage is applied to the surface every fifteen or twenty minutes by means of troughs, nozzles, or traveling distributors so as to spread uniformly in a fine spray over the entire surface, Fig. 60. It then trickles down through the broken stone, only a few minutes being required for it to pass through the filters. The period of application is usually five minutes. The bottom of the filter is entirely underlaid with tile to assist in aëration, and provision is usually made for ready access to the underdrains and distributors, for frequent inspection and for cleaning if necessary.

Sprinkling filters are operated in America under ordinary conditions with clarified sewage at a rate of two and one-half million

Fig. 61. Danville, Pennsylvania, Sprinkling Filters in Operation at 14 Degrees below Zero

gallons per acre per day. Here the usual method of distributing the sewage on to sprinkling filters is by fixed nozzles connected by piping system to an automatic siphon, or to a motor-driven rotating valve. This siphon or valve is supplied by a dosing tank of capacity sufficient to furnish a five-minute dose to the filter and permit the desired resting period. These tanks are usually built in the form of an inverted cone and are known as tapered tanks, this arrangement being necessary to distribute uniformly the sewage over the area supplied by each fixed nozzle. The flow line in this tank is from 5 feet to 10 feet above the surface of the filter in order to give the pressure necessary for distribution. It is, therefore, essential to have a total head of at least 11 feet for proper operation of sprinkling filters with fixed nozzles.

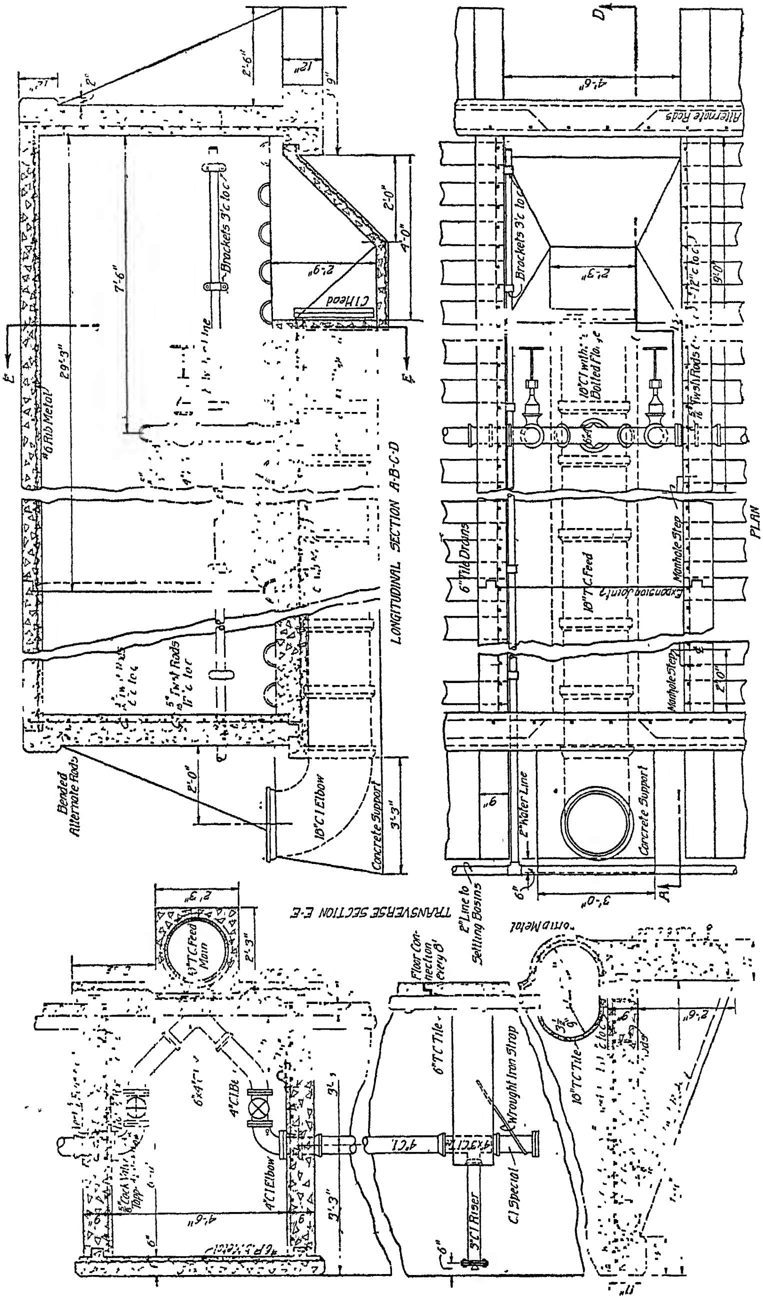


Fig. 62. Typical Details of Sprinkling Filters

148. Results. The effluent from filters of this type is more putrescible and usually shows a marked reduction in bacterial count over the raw sewage. These filters are self-cleansing, freeing themselves of the accumulated material after it has been mineralized. The effluents are, therefore, not clear, but this suspended material may be removed by settling.

149. Examples. Figs. 60 and 61 show sprinkling filters in operation for the Polk Plant and the Danville Plant. The Danville filters were being operated at a temperature of fourteen degrees

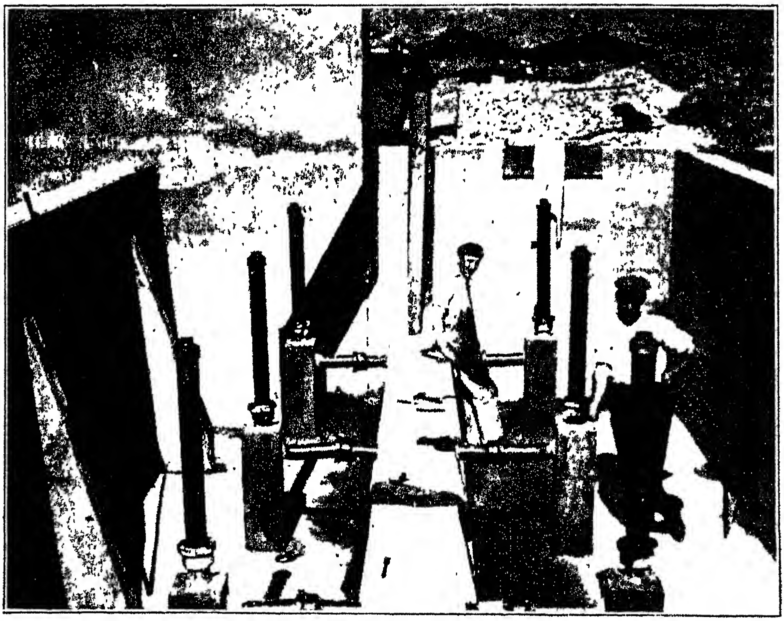


Fig. 63. Interior of Small Sprinkling Filters, Showing Distribution System

below zero at the time this picture was taken. No trouble has been experienced in America in operating sprinkling filters during cold weather.

150. Polk Filters. Figs. 62, 63, and 64 show details of the construction of sprinkling filters. It will be noted that all of the filters are operated by automatic siphons supplied by concrete tanks at the outlet ends of the settling tank. These siphon chambers are designed for a capacity sufficient to give an interval of fifteen to twenty-five minutes between doses as may be desired.

To refer in detail to the Polk sprinkling filters, which are typical of American installations, these filters are designed in two units, separated by a concrete gallery in which are located the valving and connections for the distributing line. This gallery is 4 feet wide and 6 feet deep. Under the floor of this gallery there is an 18-inch conduit connected to the siphon chamber at one end of the gallery and adjacent to settling tanks. From this conduit

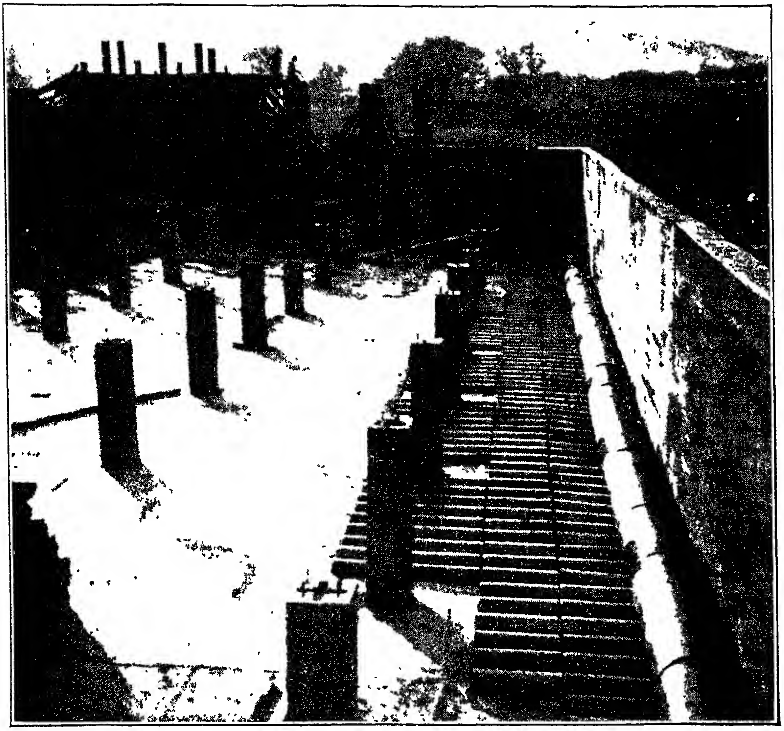


Fig. 64. Interior of Sprinkling Filter Showing Underdrains Partly Laid

at intervals of 12 feet there are 6-inch risers which supply the 4-inch distributing lines in the two filters. Each distributing line is controlled by a 4-inch valve. The distributing lines, as will be noted, are constructed of cast iron and are supported every twelve feet by concrete columns. Along these distributing lines are located the riser pipes to the nozzles, which are so spaced as to place the nozzles 14 feet center to center. The distributing lines can be

cleaned readily by removing the flanged elbows in the operating gallery. The nozzles consist of a brass throat $\frac{9}{16}$ inch in diameter, above which is set a brass cone, as shown in Fig. 65. The sewage when discharged through the throat of the nozzle, strikes this cone and is sprayed over the surface of the filter as indicated in the illustration. The underdrains for these filters consist of 6-inch split tile laid on a concrete slab and sloping from the control gallery to a cross-drain located at the opposite side of each filter. These underdrains are spaced 12 inches center to center and extend through the gallery wall to the interior so that they can be flushed out with a hose from the gallery and can also be ventilated through same. The filtering material consists of a hard sandstone ranging in size from 4 inches to 1 inch with an effective depth of 6 feet. These filters are constructed with concrete walls around each unit. In many installations, however, a dry rubble wall is used for the outside walls and in some cases where the filters are located above ground, the filtering material itself is carefully laid up to serve as a wall for the filter.

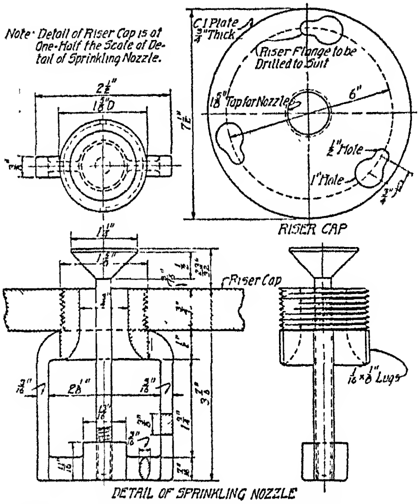


Fig. 65. Details of Sprinkling Filter Nozzle

SAND FILTERS

151. Use. Sand filters represent an artificial type of broad irrigation in two respects: (1) they consist of specially prepared filters of coarse sand, well underdrained and provided with distributing troughs for applying the sewage uniformly to the surface; and (2) the results obtained are from the action of *aërobic* bacteria on the organic and suspended matter that is retained in the sand. They may be used to treat raw sewage or clarified sewage or as a final treatment after biological filters. Sand filters will give a

much higher degree of efficiency than the two types of biological filters previously described, and the effluent from sand filters should not only show a high degree of efficiency in the removal of organic matter, but also a marked reduction in the bacteria.

152. Design. Sand filters require an area of $1\frac{1}{2}$ acres per 1000 people for raw sewage; $\frac{1}{2}$ to 1 acre per 1000 people for clarified sewage; and $\frac{1}{8}$ to $\frac{1}{4}$ acre per 1000 people as a final for biological filters. They must be operated intermittently to permit a thorough aëration of the sand between the treatment periods. These filters are usually constructed of a depth of from 2 feet to 4 feet and are underlaid with underdrains in a manner similar to the contact filters. The sewage is usually applied to a depth of 2 inches over the entire surface of the filter at intervals of not less than 8 hours apart and at longer intervals if possible. The sewage is usually distributed by a series of wooden troughs extending over the surface of the filter from the automatic control apparatus and provided at frequent intervals with gates or notches for spreading the sewage over the surface. In cold climates a ridge and furrow method of distribution must be used to prevent freezing.

Sand filters are usually installed in groups, arranged and controlled in a manner similar to that described for contact beds, with the exception that the sewage is applied to the trough system on the surface, there being no control to the outlet drains.

153. Alliance Filters. Fig. 66 shows the arrangement of sand filters which are used to filter the effluent from the contact beds at Alliance, Ohio. Sewage is allowed to run onto one bed at a time from a common collector connected with the contact beds. After several hours, the flow is shut off by hand and turned onto the next bed. These beds, with a uniform depth of three feet, have a total area of four acres and are therefore designed to treat the contact bed effluent at a rate of 500,000 gallons per acre per day. Having been distributed over the surface by wooden troughs, the sewage is collected by a system of terra cotta tile drains as shown. A central control tank is also frequently installed of a capacity sufficient to dose out enough sewage to cover the surface of the filter to a depth of 2 inches and to store enough to give the proper periods of intermission, as above outlined.

Sand filters in most locations in America call for an installation

expensive in comparison with that of contact beds or sprinkling filters supplemented by disinfection, and they are therefore few

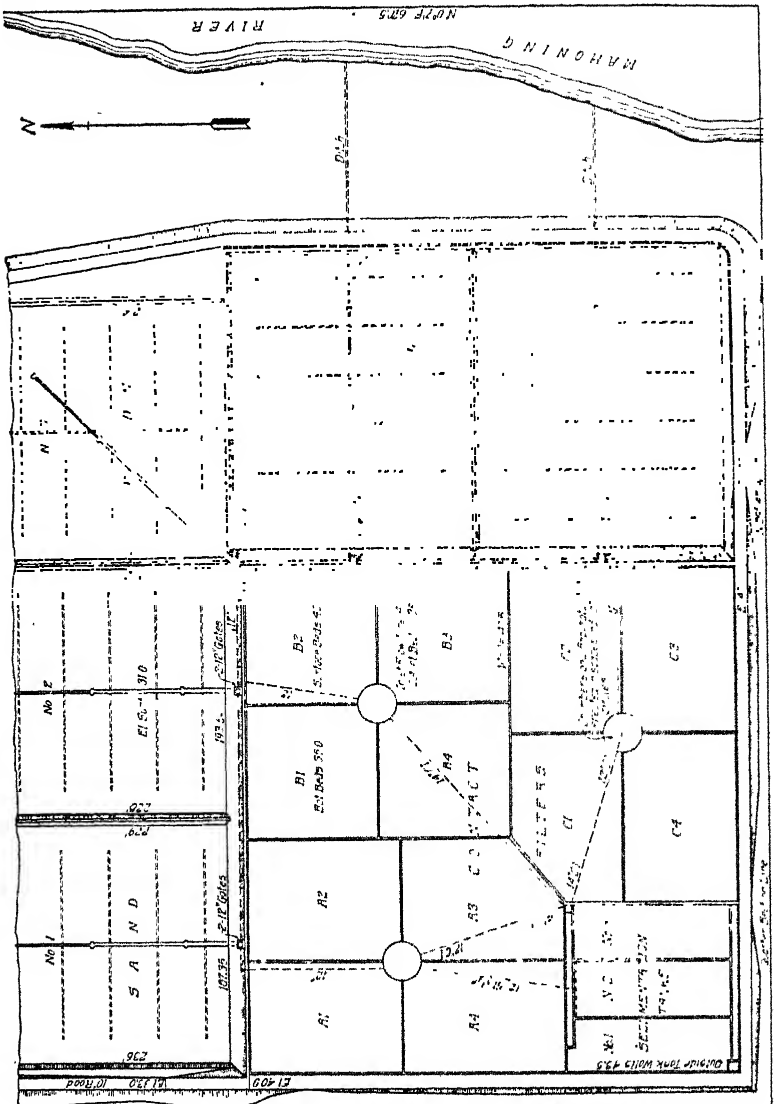


Fig. 66. Plan of Sand Filters at Alliance, Ohio

in number on any scale, outside of the New England district where sandy areas are abundant:

DISINFECTION

154. Purpose. Where sewage or the effluent from a disposal plant is discharged into a stream above a water supply, it is necessary to remove the danger of transmitting water-borne diseases. This is accomplished by disinfection or the removal of pathogenic bacteria as indicated by the absence of *b. coli.* Sterilization consists in the removal of all bacteria. As the pathogenic bacteria are weaker than the rest of the sewage bacteria, they are removed more easily and at less expense. It is found also that in sterilizing sewage, the organic matter is unaffected and upon its being discharged into a stream, new growths of bacteria will quickly develop from those already in the stream. Sterilization is, therefore, usually not attempted.

155. Method. Disinfection may be accomplished by the application of a definite amount of hypochloride of lime in solution, or chlorine gas, or electrolytic action. Chloride of lime may be purchased commercially at from $1\frac{1}{4}$ cents to 3 cents per pound depending on the size of containers, with a rated strength of 33 per cent available chlorine. It is then dissolved in a weak solution of water, and this solution is applied to the sewage at a rate of 3 to 4 parts available chlorine per million parts of sewage by weight, or 75 to 100 pounds per million gallons. At this rate, a complete removal of *b. coli* can be obtained and a great reduction in total bacteria. Most of the bacteria are destroyed instantly, but it is necessary to have fifteen or twenty minutes' contact to obtain thorough disinfection. After treatment, the sewage is therefore allowed to flow through a compartment of fifteen or twenty minutes' maximum flow capacity.

The chlorine gas treatment costs about the same as the chloride of lime and is more efficient in that it is more easily controlled. It consists of containers of chlorine gas compressed to a liquid. This liquid is fed by automatic apparatus at the desired rate into the sewage.

The electrolytic treatment is more expensive in operation under ordinary conditions. It consists in passing a current through the sewage between poles which form an electrolyte and give a nascent oxygen treatment to the sewage. In addition to the disinfection, there is a precipitation of a part of the suspended matter.

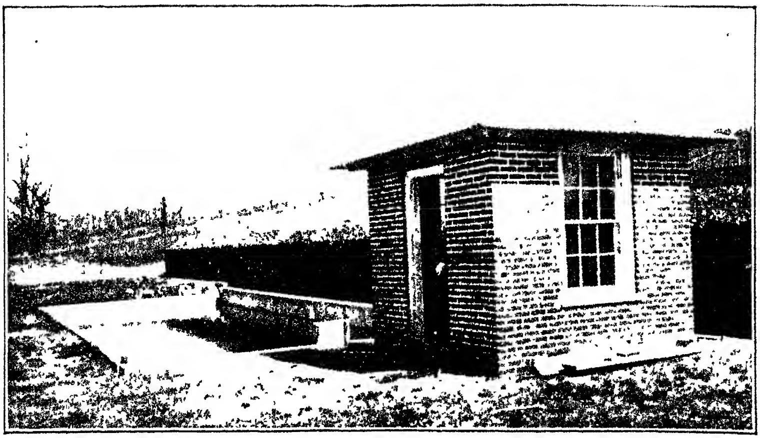
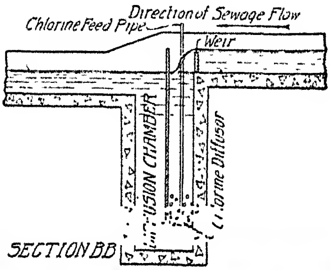
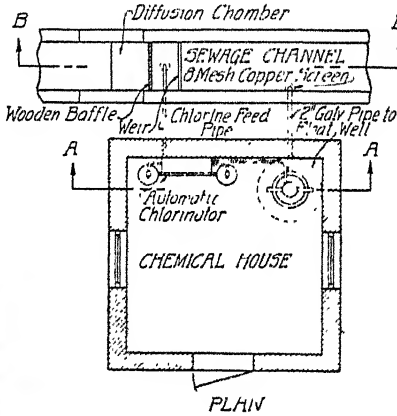


Fig. 67. General View of Liquid Chlorine Disinfection Plant





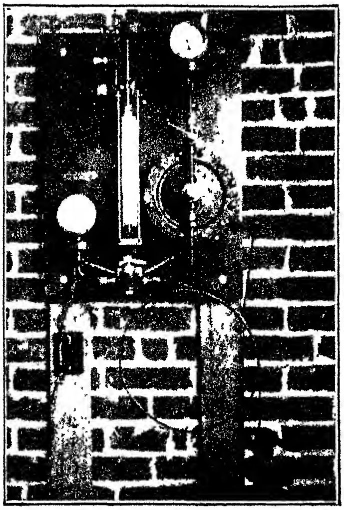


Fig. 68. Automatic Chlorinator

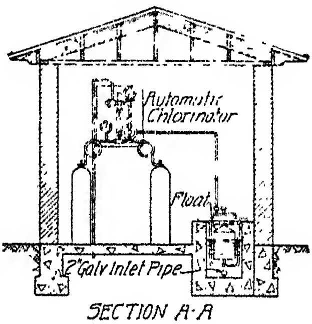


Fig. 69. Plant and Sections of Disinfection Plant

156. Automatic Appliances. Automatic devices are required for efficient disinfection since it is necessary to apply the solutions at a rate in proportion to the flow of sewage, and as the flow varies hourly. With the use of chloride of lime, this can be accomplished by a float chamber connecting to the sewer and arranged so that the float operates a lever which varies the head on the orifice from the solution box in proportion to the flow of sewage. A diaphragm control from a Venturi meter on the sewer is another method. Liquid chlorine must be fed by automatic apparatus that not only keeps the treatment of the sewage uniform, but also controls the liquid chlorine pressure tank.

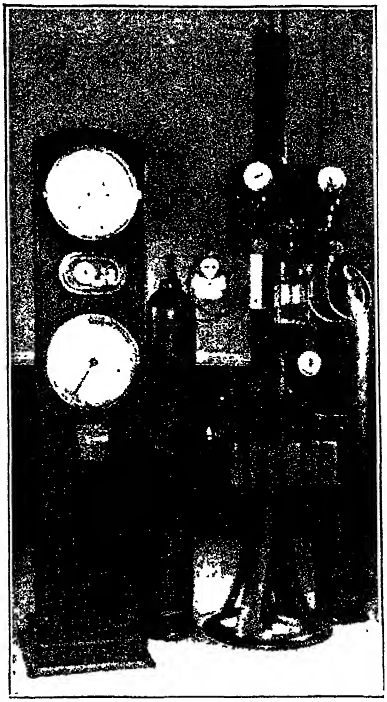


Fig. 70. Automatic Control Apparatus for Liquid Chlorine

157. Chlorine Gas Installation. Figs. 67 to 69 show a typical installation of a liquid-chlorine plant for treating the effluent from a sewage disposal plant. This plant consists of a small brick building, Fig. 67, in which the automatic apparatus is located and a re-settling basin of reinforced concrete which permits the sewage to come in contact with the liquid chlorine for a minimum period of fifteen minutes before being discharged into the stream. The automatic apparatus, Fig. 68, is controlled by a float chamber, Fig. 69, located in one corner of the pump room and directly connected to the sewer outside the building at a point on the upstream side of a fixed weir. This float transmits the difference in elevation, due to the varying flow of sewage over the weir, by diaphragms directly to the central control diaphragm. Here the drop in pressure across the chlorine gas constriction in the connecting valve from the chlorine-pressure tanks is kept proportional to the head

on the weir. This gives a proportional flow of chlorine for the varying heads over the weir. The liquid chlorine is then fed to the outfall sewer below the weir and is liberated in a baffled concrete compartment called a "diffusion chamber", section *BB*, so that the chlorine will be thoroughly mixed with the sewage to be treated.

Fig. 70 shows another type of liquid-chlorine apparatus where a different type of control is used. This consists of pressure regulating devices to produce the initial cylinder pressure of the liquid chlorine and to control this pressure through a range sufficient to give the required discharge of gas. On the outlet line, there is attached a low-pressure chlorine gauge calibrated to indicate the rate of flow of chlorine gas. This apparatus can also be used in connection with Venturi meters to supply the chlorine automatically in proportion to the flow of sewage.

158. Results. Where the chlorine-gas apparatus is used on raw sewage, the results are unreliable if there are large pieces of organic matter in suspension, as the treatment will not destroy the bacteria inside these masses. For treatment of clarified sewage or effluents from disposal works, a uniformly high degree of efficiency can be obtained.

PUMPING

159. Requirements. Where it is at all possible to eliminate pumping, it should be avoided, as it not only adds a high operating cost, but also is difficult in maintenance. Pumping is, however, a necessity at many disposal works where the outfall sewer is too low to permit of gravity operation or where the plant would otherwise be subjected to flood conditions. It is necessary, too, for many of the modern buildings in our large cities where the deep basements and sub-basements are entirely too low to drain to the sewers.

160. Method. Sewage pumps must be reliable, free from obstructions, and in many cases automatic. Centrifugal pumps with open type impellers, steam or motor driven, are naturally adapted to sewage pumping on account of simplicity, possibility of automatic operation, and their large capacity for low heads. Automatic ejectors operated by compressed air are usually used on small installations.

161. Design. For stations that are to be automatic, motor-driven centrifugals are well adapted. The pump must be submerged and preferably placed in a compartment separate from the sewage so as to be accessible at all times. Particular attention must be paid to protecting the pumps from injury or clogging from suspended matter in the raw sewage, and for this purpose, screen chambers must be designed and installed as previously outlined under that heading. The motor should be placed on the operating floor above the pump well to prevent trouble from dampness.

The pump well must be of capacity sufficient to permit the sewage to reach the pumps without swirling and also to give some margin for a temporary closing down or changing over of pumps, and, in the case of automatic stations, to permit of proper periods between stopping and starting. A minimum of thirty minutes' storage of the maximum flow should be used. In determining the size of the pump, study must be made of the varying flow of sewage at different hours of the day, and under maximum and minimum conditions; while its capacity must be such that it can take care of the flow of sewage under all conditions and at the same time permit of at least one pumping unit being out of operation. Where there is a possibility of the supply of electric current being interfered with, auxiliary power should be provided.

In the design of larger stations or of stations where the cost of current is not comparable with other fuel, steam-driven or gas-engine-driven installations can be made, but stations of this type must be supplied with operators.

162. Connellsville Pumping Station. Fig. 71 shows a plan and section of an automatic electric-driven pumping station designed for Connellsville, Pennsylvania. This station consists of a circular pit 50 feet in diameter by 30 feet deep, with the top of the pit carried up to the natural ground level, which is above extreme high-water mark, and with the bottom of the pit 7 feet below the invert of the connecting sewer. The pumping equipment consists of two 7-inch centrifugal pumps run by electric motors and each having a capacity of one and one-half million gallons per twenty-four hours; and one 10-inch auxiliary pump run by a gas engine and having a capacity of three million gallons per twenty-four hours. There is a concrete diaphragm wall separating the pump pit from

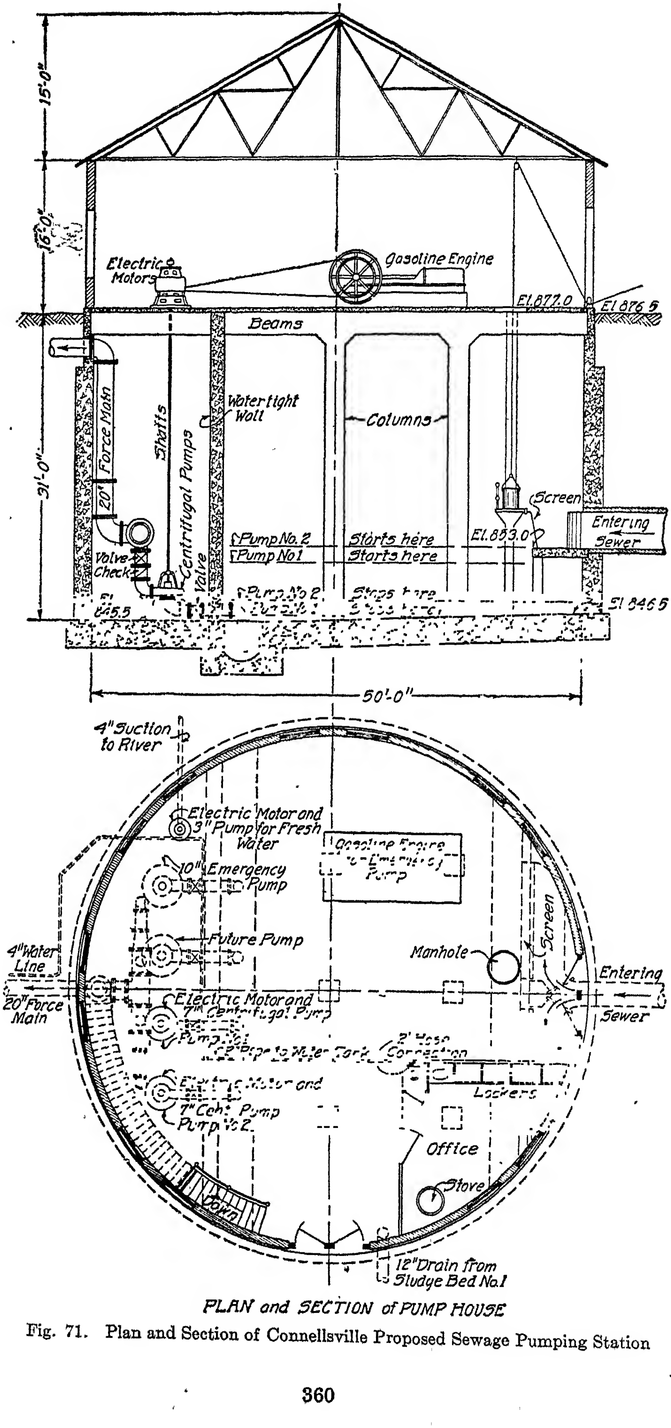


Fig. 71. Plan and Section of Connellsville Proposed Sewage Pumping Station

the suction pit so that the pump pit will be dry at all times and pumps can be conveniently reached for inspection or repairs. The main suction pit will have an effective capacity of 75,000 gallons or a minimum storage of 30 minutes under maximum conditions. The motors operating the pumps will be located on the reinforced-concrete operating floor above the pit and directly connected to the pumps by vertical steel shafts. Each motor will be automatically controlled by a float located in the pit and connected to the switchboard so that when the sewage rises in the pit to a depth of four feet, one pump starts up. If the flow is greater than the capacity of this pump, the sewage will continue to rise for another foot when the second pump will start up. When the sewage drops in the pit to within two feet of the bottom, the last pump in operation is cut off by the float and when it reaches the bottom, the second pump is placed out of commission. The auxiliary pump is provided to take care of any breakdown in the other machinery and also to provide against a failure of the current supply. A small 3-inch pump similar to the other electric-driven pumps is provided for taking raw water from the river and supplying it for flushing purposes through a force main to the disposal works.

163. Ejectors. Figs. 72 and 73 show an installation of Shone ejectors and also a sectional detail of one of these ejectors. Ejectors like pumping installations, should be set up in duplicate as indicated in the illustration, so that there will be a spare unit for use in emergency. Ejectors are simple in operation as they have no working parts which can become clogged with suspended matter, and when operated with compressed air, can be located at a considerable distance from the air compressor or air tank.

As will be noted in Fig. 73, these ejectors consist of a closed cast-iron vessel furnished with inlet and outlet connections which are controlled by check valves. Gate valves are also provided, but are only used to disconnect one unit for repairs. On top of the ejector is placed an automatic valve to which is connected the air pipe from the air compressor. This valve controls the admission of the compressed air into the ejector and also the exhaust of displaced air from the ejector. It is operated by the two cast-iron bells hung in reversed positions and linked to each other by a rod through the center of the main compartment as shown. When

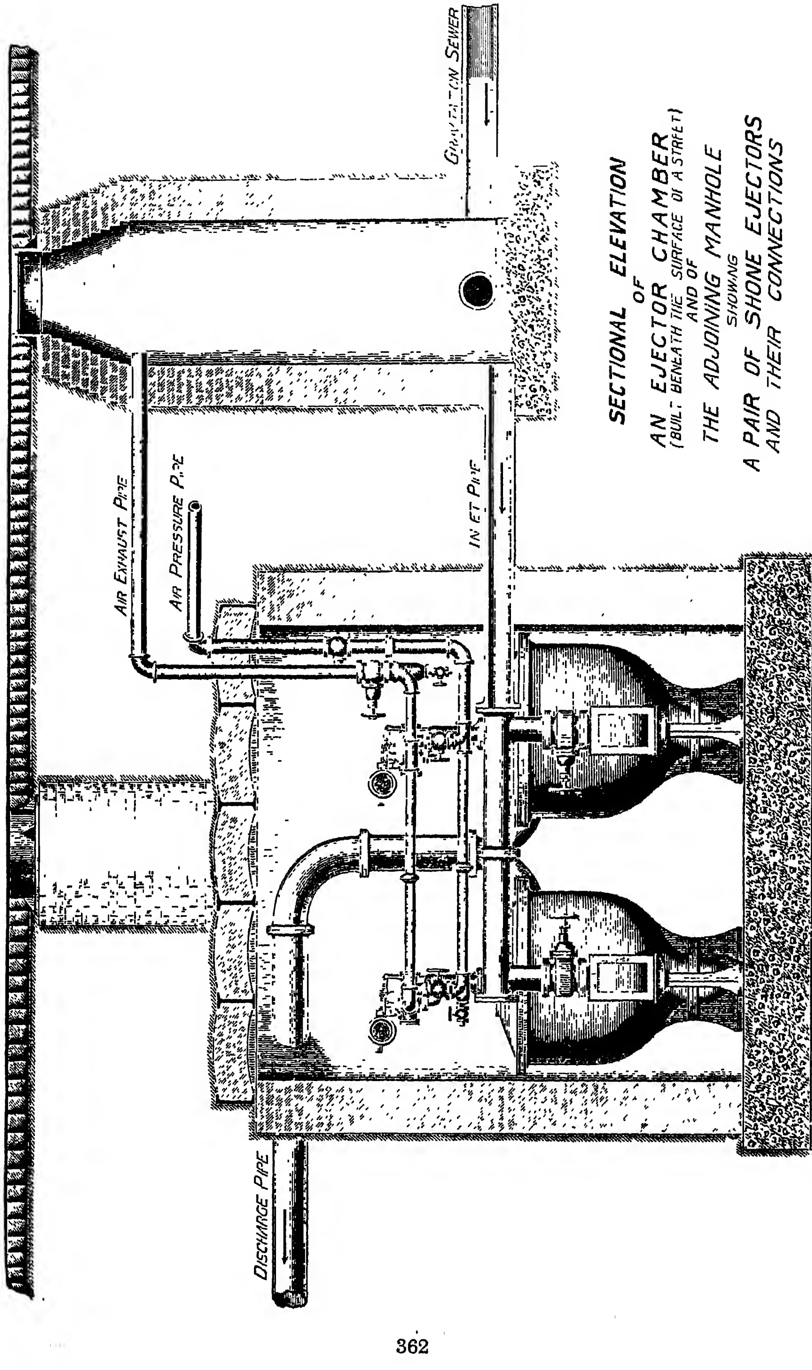


Fig. 72 Typical Installation of Shone Ejectors

the sewage rises in the ejector, the exhaust valve is opened and the air valve is closed, the bells being in the lower position. When it reaches the top of the ejector, it traps air in the upper bell forcing it to rise by buoyancy. The lever connected with the rod from the bells quickly closes the exhaust and opens the compressed-air

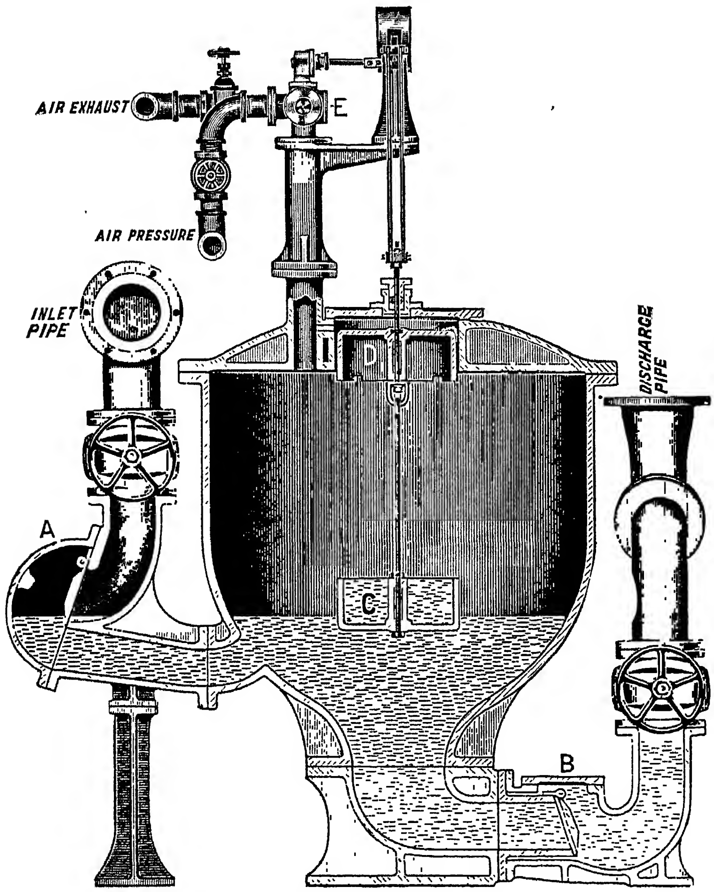


Fig. 73. Detailed Section of Shone Automatic Ejectors

connection and the compressed air immediately forces tne sewage from the compartment into the discharge pipe. When the sewage reaches the lower bell, its weight pulls down the control rod, thereby reversing the position of the pressure and exhaust valves and allowing the ejector to start again the process of filling. It requires

very little more air pressure to operate this type of ejector than the head of water against which it must lift.

SUMMARY

164. Sewage-Disposal Method—Question of Conditions. It will be seen from the outline given of the various methods of sewage disposal and the results that can be obtained, that each case must be studied and worked out as an individual problem. Dilution is feasible under certain conditions, but usually with some treatment such as screening or disinfection, or a partial removal of the suspended matters. Broad irrigation and chemical precipitation, which were once popular, are now installed only under exceptional conditions on account of the high cost of these methods of treatment. A non-putrescible effluent with a high percentage of removal of the organic matter and bacteria can be obtained by broad irrigation, but only a partial removal of it can be obtained by chemical precipitation.

165. Care of Suspended Matter and Effluent. For the removal of suspended matters, mechanical screens, Imhoff tanks, and septic tanks are generally used. Where there is danger of trouble from odors, screens or Imhoff tanks are preferable. Where it is necessary to produce a non-putrescible effluent, a secondary treatment must be given the effluent from which the suspended matter has been removed, and this is accomplished by biological filters of which the sprinkling filters and contact beds are the types generally used. Sprinkling filters are preferable to contact beds on account of lower cost, but require more head so that in many cases they cannot be considered.

166. Disinfection of Effluent. The effluent from biological filters is not free from bacteria and where a removal of pathogenic bacteria is required, the effluent must be disinfected. This is generally accomplished by chloride of lime or liquid chlorine. Sand filters will give a much higher degree of efficiency than the coarse-grain biological filters, but on account of high cost are not generally used.

Electrolytic treatment will efficiently disinfect sewage, but on account of cost of treatment, it has not been able to compete with the other methods of disinfection. Experiments are now

being conducted on activated sludge, formed by blowing compressed air through freshly deposited suspended matters. This process appears to liquefy and nitrify the sludge in a very short period of time by accelerating the growth of enormous numbers of small worms in the sludge deposits. These experiments indicate a development of a new method of handling the sludge problem, although on account of the high cost of operating it is doubtful whether it will be as economical as the double-story tank method with separate digestion compartment now so generally and successfully used.

Pumping should be avoided if possible. If it must be used, the apparatus must be arranged to avoid clogging and in most small installations to be automatic. Where compressed air is available, an ejector is the simplest type of pump to use.

167. Future Conditions. As the population of this country increases and new towns spring up, the necessity for purification of sewage increases and the time is not far off when even our seacoast cities must adopt partial purification. It is, therefore, of vital importance to make the present type of partial treatment now installed, whatever it may be, subject to further development of a higher degree of purification.

Finally, too much emphasis cannot be placed on the importance of a thorough study of conditions, and of development of a comprehensive scheme to embrace not only present circumstances, but future contingencies.

GARBAGE DISPOSAL

168. Introduction. The disposal of garbage, like that of sewage, has been placed on a scientific basis in recent years only. Formerly, it was common practice to dump it into rivers; to bury it in waste tracts of land; or to spread it on these tracts, leaving it to rot with the resultant stench and nuisance. In recent years, many types of incinerators and reduction plants have been developed and placed on a successful operating basis.

169. Composition of Garbage. The term garbage is generally applied to the rejected food wastes of a community, but in addition includes ashes and household rubbish, as well as street sweepings, and the offal from slaughter houses and carcasses. The average garbage contains 70 per cent water, 3 per cent grease, 20 per cent

organic matter, while the balance is miscellaneous material. The household wastes consist mainly of paper, rags, metal, glass, bottles, and crockery. Paper and rags predominate in the rubbish. It has been estimated by Craven that in the New York City rubbish, 75 per cent is paper and 15.5 per cent rags.

170. Quantity. The amount of garbage and other waste materials to be disposed of obviously differs greatly in different communities, depending to a great extent on local conditions. From data collected in several cities in America, it is found that garbage will vary from 100 to 200 pounds, ashes 300 to 1000 pounds, and rubbish from 50 to 100 pounds, per capita per annum.

171. Disposal. Household garbage has a food value for swine, but it is impossible to handle it in a sanitary way on any great scale so that only in the case of large public institutions, or very small communities, can this method of disposal be used. The grease and organic matter also have a commercial value for soap and fertilizer; the rubbish can be sorted and sold, but as in the case of the garbage, there is no economy in their sale, unless it be carried to a considerable extent. The ashes are valuable for filling in low tracts of land.

For towns under 50,000 population, the best method for disposing of garbage and rubbish is by incineration. For larger cities, a reduction in the operating cost can usually be effected by a reduction and sorting plant, or by selling it to a reduction company.

172. Collection. Whatever the method adopted, dealing with garbage, it is preferable to collect it separately from the rubbish and ashes. This makes the handling much easier, and even where all materials are to be disposed of by incineration, permits of more efficient charging and operation of the incinerator. The garbage must be collected in water-tight carts with air-tight lids and arranged for automatic dumping at the point of delivery. The other material may be collected in ordinary dump wagons.

INCINERATORS

173. Requirements. The essential requirements of a garbage incinerator are: (1) economy in operation with freedom from odors; and (2) capacity sufficient to take care of the entire garbage supply. It is necessary first, therefore, to know the character and amount

of material to be taken care of. It is obvious that to burn garbage only will require a design of furnace radically different from that used to burn it with rubbish or ashes, or both. Where garbage alone is burned, a greater amount of fuel is necessary, while drying grates and evaporating pans must be provided to prevent the liquids from reaching the main grate bars. After determining the amount and character of the garbage to be burned, the operating periods must be determined. A continuous operation is economical from the fuel standpoint, but for small plants, it would make the labor cost too high, as one man on one shift can easily handle eight tons of material.

174. Design. The design of garbage crematories has been developed by a great number of companies, and the patents covering the various features are legion. To prevent odors, it has been found that a temperature of 1200° F. must be reached in the gases in the outlet flues of the plant, and this requirement, together with economy of fuel consumption and permanence of grate bars and fire linings, is the controlling feature. A good furnace builder could design an incinerator that would avoid the various patents, but it is doubtful whether this design would also incorporate all of the desired features for economy and permanence.

It is, therefore, better for the engineer, who is planning to install an incinerator, to prepare a general plan and specification for the work and to permit the various companies to submit bids on their own designs, conforming to the requirements of the general plans and specifications. Having received these bids, the engineer should carefully study the various features of each design and investigate the results that have been obtained in operation with particular attention to efficiency and replacement charges. A high first cost is in many cases justified by the saving effected in maintenance.

REDUCTION PLANTS

175. General Conditions. In disposing of the garbage for cities of 50,000 population or over, a considerable saving in the operating cost can be effected by so reducing the garbage as to obtain the fats and fertilizing materials. If the rubbish is collected separately, it will also pay to install a sorting department.

As before stated, the sorting of rubbish, like the reduction of garbage, is not economical unless large quantities are handled. The waste must be subdivided into a great many parts which requires a large operating force, as one individual can handle only two or three parts. The sorting is usually done in a large room; through the center of this there is a belt conveyor, along which are distributed the operating force who pick off from the conveyor the various articles to be sorted as they pass.

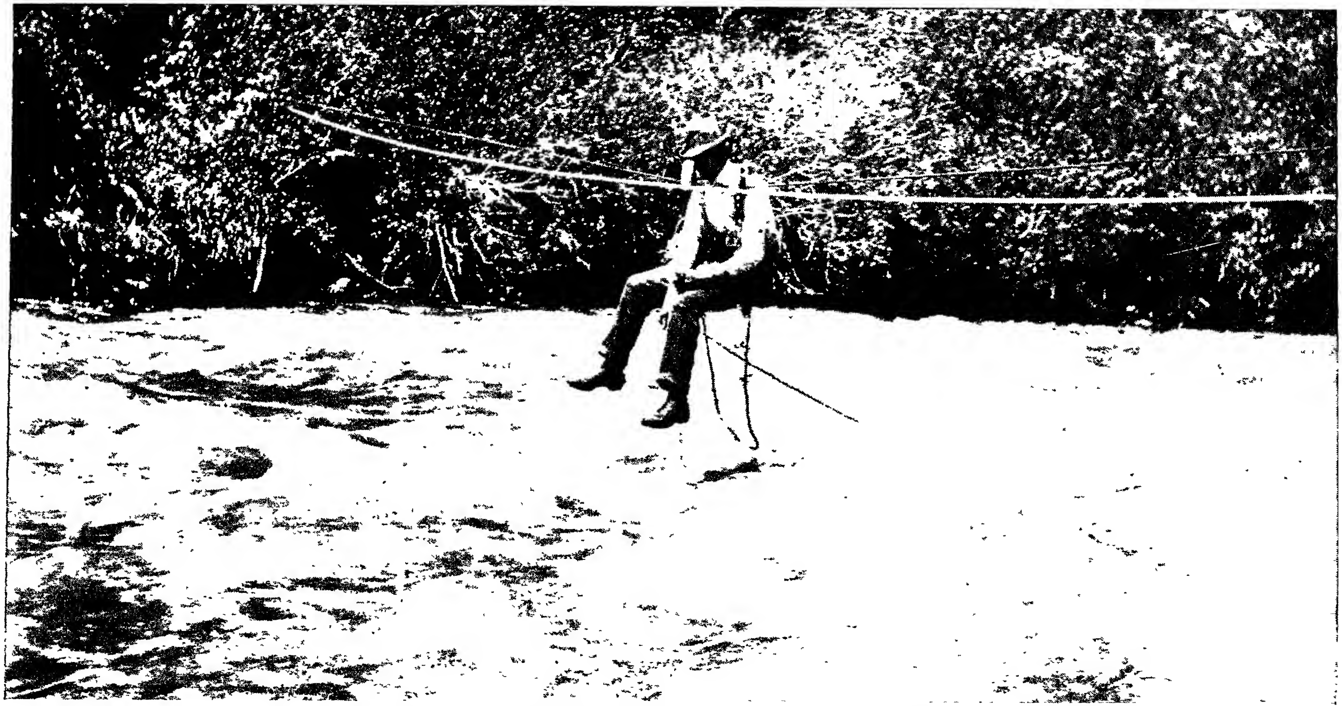
The reduction plants are in most cases installed and operated by a private company which makes a contract with the city for disposing of the garbage. There are, however, several municipal reduction plants which have been installed by the cities and operated by them successfully.

176. Columbus Reduction Plant. The garbage reduction plant of Columbus, Ohio, is a typical one. It was installed by the city in 1910, for the reduction of all of its garbage. The plant consists of the boiler plant and machine shop, and also digesters, presses, grease-separating tanks, percolators, refining and storage tanks, drying equipment, screens and evaporators. The garbage is reduced to obtain grease, fertilizer, and hides, the last of course, being stripped before the carcasses are placed in the reduction plant proper.

The method of operation consists in first draining off the liquids from the garbage to tanks; here the grease is separated by gravity whence it is pumped to treating tanks. After the grease has been removed, the water is pumped to an evaporator and concentrated to a syrup to recover the solids in solution. The drained garbage discharged into large digesters filled with steam is thoroughly cooked. The odors or gases from the digesters are passed through condensers and the insoluble gases through deodorizing furnaces. The cooked garbage is next subjected to presses which remove the solids from the liquids. The liquids are drawn off to the grease-separating room, the solids being carried to a series of driers. The solids are then treated in sealed containers with gasoline, which acts as a solvent and separates what grease has been retained in them. The gasoline is removed by means of dry steam; the water, grease, and gasoline are separated and returned to their respective storage tanks. The solids having been mixed with the tankage left over

from the evaporators so as to absorb all solids contained therein, are now dried and sold for fertilizer.

The operating report for this plant in 1912, showed that an average of 60 tons of garbage was treated per day, the total cost of operation of the plant \$38,500, and the total receipts from the products \$61,700. As the plant cost \$210,000, it is necessary to add to the cost of operation \$15,500 for interest and sinking fund. This leaves a balance of \$7,700 profit. The above outlined cost of operation does not, however, include the collection of the garbage in the city nor its delivery to the disposal works.



HYDROGRAPHER GAUGING A WILD MONTANA STREAM

HYDRAULICS.

1. Hydraulics is that branch of Mechanics which treats of the laws governing the pressure and motion of water. *Hydrostatics* is that particular branch of hydraulics which treats of water at rest, and *hydrodynamics* is that branch which treats of water in motion.

2. Units of Measure. The unit of length most frequently used in hydraulics is the foot. The unit of volume is the cubic foot or the United States gallon. The unit of time usually employed in hydraulic formulas is the second, but in many water-supply problems the minute, the hour, and the day are also often used. The unit of weight is the pound, and that of energy the foot-pound.

1 U. S. gallon = 231 cubic inches = 0.1337 cubic foot;

1 cubic foot = 7.481 U. S. gallons;

1.2 U. S. gallons = 1 Imperial gallon.

3. Weight of Water. The weight of distilled water at different temperatures is given in Table No. 1.

The weight of ordinary water is greater than that of distilled water on account of the impurities contained. For ordinary purposes the weight of a cubic foot of fresh water may be taken equal to 62.5 pounds. Sea water will weigh about 64 pounds per cubic foot.

TABLE NO. 1.

Weight of Distilled Water.

Temperature, Fahrenheit.	Weight, Pounds per Cubic Foot.	Temperature, Fahrenheit.	Weight, Pounds per Cubic Foot.
32°	62.42	140°	61.39
39.3	62.424	160	61.01
60	62.37	180	60.59
80	62.22	200	60.14
100	62.00	212	59.84
120	61.72		

As will be seen from this table, water is heaviest at a temperature of about 39.3° F., or as is commonly stated, about 40° F.

4. Atmospheric Pressure. As has already been explained in the papers on Elementary Mechanics, the atmosphere everywhere exerts a pressure upon all objects uniform in every direction, and is itself compressed to the same degree. At sea level the average pressure of the atmosphere is sufficient to balance a column of mercury in a closed tube (a barometer) about 30 inches high, which is equivalent to a pressure of 14.7 pounds per square inch. A corresponding water barometer would be 34 feet high, the weight of water being much less than that of mercury. At points higher than sea level the air pressure is less, and hence the height to which a mercury or water barometer will be raised will be less. Since we depend upon air pressure to raise water into "suction" pipes it is important to know how much this pressure is when designing such pipes.

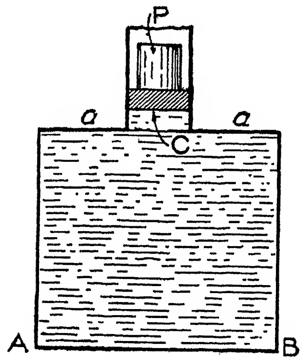


Fig. 1.

The following table gives, for different elevations above sea level, the pressure of the atmosphere, expressed, first, in pounds per square inch, second, in the height of the mercury barometer and, third, in the height of the water barometer:

TABLE NO. 2.

Atmospheric Pressure at Different Elevations.

Elevation above Sea Level. Feet.	Pressure in Pounds per Square Inch.	Height of Mercury Barometer. Inches.	Height of Water Barometer. Feet.
0	14.7	30.00	34.0
500	14.5	29.47	33.3
1,000	14.2	28.94	32.8
2,000	13.7	27.92	31.6
4,000	12.7	25.98	29.4
6,000	11.8	24.18	27.4
8,000	11.0	22.50	25.5
10,000	10.3	20.93	23.7

PRESSURE OF WATER AT REST.

5. Transmission of Pressure. If AB, Fig. 1, be a tight vessel containing water, and a close fitting piston C be heavily loaded with a weight P the entire body of water will be subjected to a pressure corresponding to the weight P. The water will not be compressed into a smaller space as would a gas like air, because water is almost incompressible, but whatever pressure is exerted by the weight P will be transmitted through the water equally in all directions so that the pressure of the water against the walls of the vessel will be the same per square inch as that of the weight P upon the water (neglecting the small effect of the weight of the

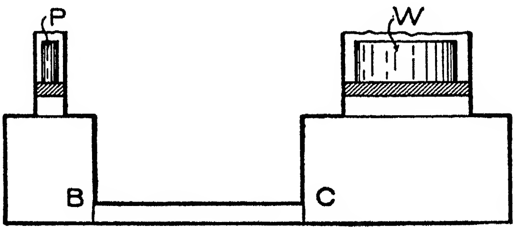


Fig. 2.

water in the vessel). Thus if the area of the piston = 10 square inches and the weight P = 1,000 pounds, the pressure per square inch will be 100 pounds, and this will be the pressure in every part of the liquid and upon the walls of the vessel. Furthermore, the pressure of the water upon the walls of the vessel is perpendicular to the surface at all points. The pressure at *a* is upwards, on the bottom of the vessel it is downwards and on the sides it is horizontal.

As a further illustration of the foregoing principle, let B and C, Fig. 2, be two vessels connected by a pipe, and let P be a loaded piston exerting a heavy pressure in the small vessel B. In accordance with the principle above stated, this pressure will be transmitted equally to the larger vessel where the water will exert the same pressure per square inch upon the vessel and upon any piston W which may be inserted in any opening of the vessel C. By making the area of P small and of W large a small load P will balance a large load W.

If a is the area of the piston P and A that of the piston W, then the pressure per square inch produced by the weight P will be $\frac{P}{a}$. This will also be the pressure per square inch on W, and

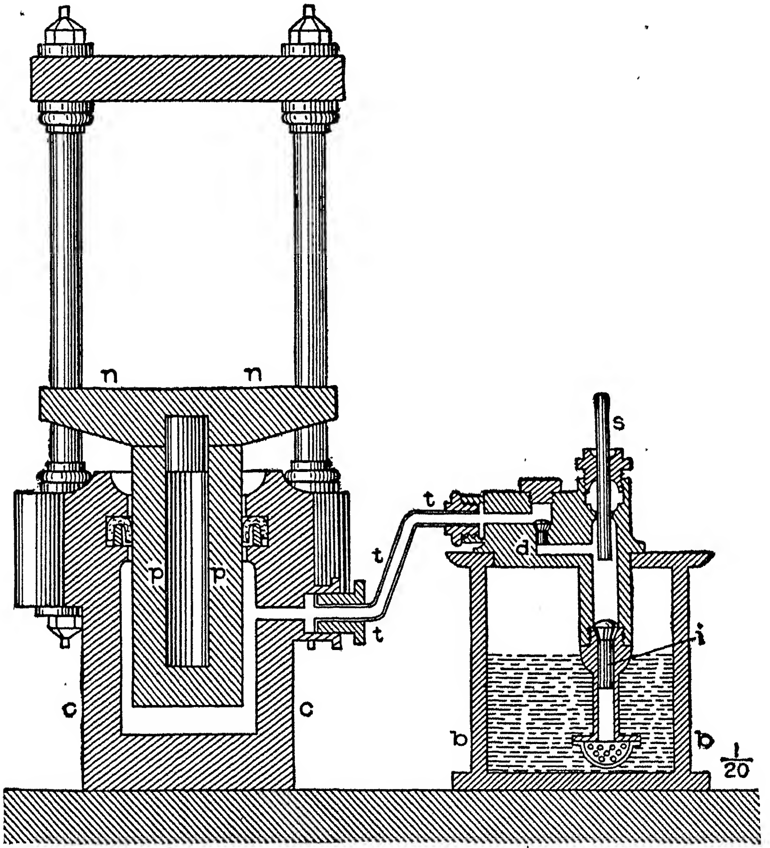


Fig. 3.

hence the weight W which will be sustained will be equal to the area A multiplied by the pressure per square inch, $\frac{P}{a}$, or

$$W = A\frac{P}{a}. \qquad (1)$$

The principle above stated is utilized in the hydraulic press shown in Fig. 3. In this apparatus a pump on the right with

small plunger feeds a large plunger p underneath the movable plate of the press on the left. The pump plunger corresponds to the piston P in Fig. 2, and the press to the piston W. By making the pump very small and the plunger under the press very large, enormous pressures can be exerted even by means of a hand pump. The pressure produced is given by formula (1) above. It is to be noted that the pressure per square inch on the interior of the apparatus, the pump, piping and press, is the same at all points.

Examples. 1. If the area of the pump plunger be 2 sq. in. and that of the press 1 sq. ft., what pressure will be exerted by the press when the load on the pump is 100 lb.?

Using equation 1 we have $a = 2$ sq. in., $A = 144$ sq. in., and $P = 100$ lb., whence $W = 144 \times \frac{100}{2} = 7{,}200$ lb. Ans.

2. If a pressure of 10 tons be desired and the area of the press plunger be 200 sq. in., and the available pressure on the pump plunger be 150 lb., what area must be given to the pump plunger?

Here $W = 10 \times 2{,}000 = 20{,}000$ lb., $A = 200$ and $P = 150$. Using equation 1 and letting $x =$ desired area, we have $20{,}000 = \frac{200 \times 150}{x}$. Solving for x we have $x = \frac{200 \times 150}{20{,}000}$
$= 1.5$ sq. in. Ans.

6. Pressure Due to the Weight of Water. Let Fig. 4 represent a vessel of water. Consider a vertical column of the water of height h and a cross-section of one square foot. Its volume will be h cubic feet and it will weigh $62.5 \times h$ pounds. As it is supported entirely by the water underneath, it therefore exerts a pressure upon that water of $62.5 \times h$ pounds. Likewise the pressure at any other point in the vessel at a distance h below the surface is $62.5 \times h$ pounds per square foot. Furthermore, since the water exerts equal pressures in all directions it follows that the pressure against the sides of the vessel at this depth, or against any object immersed in the water, will also be $62.5 \times h$ pounds per square foot.

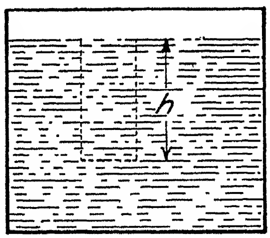


Fig. 4.

Since the weight of water is so nearly constant we may conveniently use the depth h as a measure of the pressure. When so used it is called the *pressure head* or simply the "head" acting on the given surface. For each foot of head the pressure will be 62.5 pounds per square foot, but in expressing pressure in pounds it is customary to use the square inch. A pressure of 62.5 pounds per square foot being equal to $\frac{62.5}{144}$ or .434 pounds per square inch, it follows that one foot of *head* gives a pressure of .434 pounds per square inch. Conversely, a pressure of one pound per square inch requires a head of $\frac{1}{.434}$ or 2.304 feet.

Rule. *To convert feet of head to pounds pressure multiply by* .434. *To convert pounds pressure to feet of head multiply by* 2.304. **(2)**

Examples. 1. What will be the pressure per square inch in the vessel of Fig. 5 at a point a 10 feet below the water surface? Assume the vessel to be round with a diameter of bottom = 6 feet and of upper part = 2 feet.

Here the head is 10 feet, and by the above rule the pressure per square inch = $10 \times .434 = 4.34$ pounds. It acts equally in all directions and is independent of the shape of the vessel.

2. What will be the total pressure on the bottom of the vessel?

The area of the bottom in sq. ft. $= \frac{3.14 \times 6^2}{4} = 28.26$ sq. ft. The head is 14 feet and hence the pressure per sq. ft. $= 14 \times 62.5 = 875$ pounds. The total pressure on the bottom $= 875 \times 28.26 = 24{,}728$ pounds.

3. What will be the total upward pressure on the portion AB?

The area of this portion is the difference between the two circles respectively 6 feet and 2 feet in diameter. This is equal to $\frac{(6^2 - 2^2) \times 3.14}{4} = 25.12$ sq. ft. The head is 8 feet and the pressure, therefore, $8 \times 62.5 = 500$ pounds per sq. ft. Total upward pressure $= 500 \times 25.12 = 12{,}560$ pounds.

4. What is the entire weight of water in the vessel?

The volume of the lower part of the vessel $= \dfrac{6 \times 6^2 \times 3.14}{4}$ $=$ 169.56 cu. ft., and of the upper part $= \dfrac{8 \times 2^2 \times 3.14}{4} = 25.12$ cu. feet. Total volume = 194.68 cu. ft., and weight of water = $194.68 \times 62.5 = 12{,}167$ pounds.

Note that the difference between the downward pressure on the bottom and the upward pressure on AB = 12,168 lbs., which is equal to the total weight of the water, or the net pressure of the vessel upon its support, if we neglect the weight of the vessel itself.

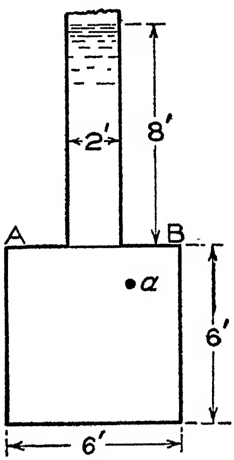


Fig. 5.

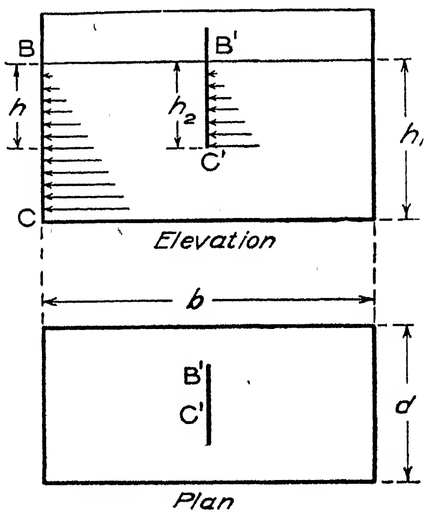


Fig. 6.

7. Pressure of Water upon Plane Areas in General. In the preceding articles it has been shown that the pressure per square inch upon any submerged body is equal to .434 h where h is the head in feet; furthermore, that this pressure is at right angles to the surface of the body. Let Fig. 6 represent a vessel of rectangular shape containing water of a depth h_1. The pressure on the bottom is then $h_1 \times .434$ pounds per square inch. If the area of the bottom be A $(= bd)$, then the total pressure on the bottom is $A \times h_1 \times .434$ pounds. In this case the pressure is the same per square inch at all points of the surface considered.

Consider now the pressure on one of the sides, as BC. In this case the pressure per square inch is not uniform, varying from nothing at B to a maximum at C where it is equal to $h_1 \times .434$ pounds per square inch, the same as on the bottom. At any depth h the pressure is $h \times .434$ pounds per square inch. This variation in pressure is represented in Fig. 6 by the variation in length of the arrows acting against BC. From an inspection of the figure it is evident that the average length of these arrows is equal to one-half the length of the one at the bottom, or in other words, the *average* pressure per square inch against BC is equal to one-half the maximum, or $\frac{1}{2} h_1 \times .434$, which is the same as the pressure at the center of BC. The *total* pressure on the entire surface is then equal to this average pressure multiplied by the total area, or equal to $\frac{1}{2} h_1 \times .434 \times h_1 d$.

If the area in question be a plate B′C′ immersed in the water to a depth h_2 the result is the same, except in this case there is an equal pressure on each side. As before, the pressure on either side of the plate is equal to $\frac{1}{2} h_2 \times .434 \times$ (area of submerged portion of plate).

If the plate be wholly submerged, as BC, Fig. 7, the pressure per square inch at B will be $h_1 \times .434$, and that at C will be $h_2 \times .434$, and the variation in pressure will be represented by a trapezoid of arrows instead of a triangle. The average pressure will now be $\frac{h_1 + h_2}{2}$ which is again the same as the pressure at the center of BC. The total pressure will be this average pressure multiplied by the area of the plate.

In all the above cases it will be seen that the average pressure found is the same as the pressure at the center of the plate. In a similar way it can be shown that for plates of *any shape* the average pressure is equal to the pressure at the *center of gravity* of the area, hence the following:

Rule. *The total pressure on a submerged vertical plane surface is equal to the pressure per unit area at its center of gravity multiplied by its area.* (3)

Suppose now the plate BC, Fig. 8, be an inclined plate immersed in water. From the principles already explained the

pressure per square inch will be the same at any given depth as if the plate were vertical. Hence at B the pressure is $h_1 \times .434$ and that at C is $h_2 \times .434$. The average pressure is again $\frac{h_1 + h_2}{2}$ $\times .434$, or the pressure at its center, and the total is equal to this pressure multiplied by the area of the plate. Whence the more general rule,—

Rule. *The total pressure on any submerged plane surface is equal to the pressure per unit area at its center of gravity multiplied by its area. Such pressure always acts at right angles to the surface.* **(4)**

8. Pressure in a Given Direction. In the above discussion we have considered only the total pressure of the water, which always acts perpendicular to the surface of the body. In Fig. 9 let P represent this total pressure on the surface BC, which has a

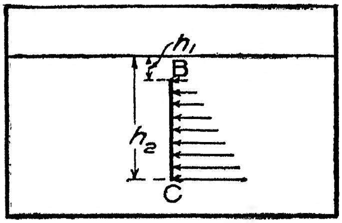


Fig. 7.

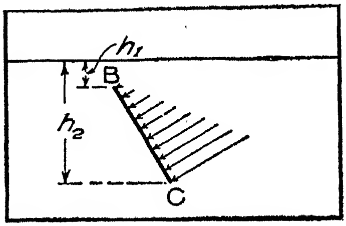


Fig. 8.

length l and a width d (its area equals ld). Suppose it is desired to find the horizontal and vertical components P_h and P_v of this pressure. Since P is perpendicular to BC the inclination of P from the horizontal is the same as that of BC from the vertical. Call this angle θ. From Mechanics we have at once, $P_h = P \cos \theta$ and $P_v = P \sin \theta$ From the foregoing articles we also have $P = h \times .434 \times ld$, in which h is the depth of the center of gravity of BC. Hence we have $P_h = P \cos \theta = .434\, h \times \cos \theta \times ld$, and $P_v = P \sin \theta = .434\, h \times \sin \theta \times ld$. From the figure we see that the area of the vertical projection of the plate BC $= m \times d = l \cos \theta \times d$, and the horizontal projection $= n \times d = l \sin \theta \times d$. Whence we have $P_h = .434\, h \times md$ and $P_v = .434$

$h \times nd$. That is, $P_h = .434\, h \times$ (vertical projection of plate), and $P_v = .434\, h \times$ (horizontal projection of plate). Whence the general

Rule. *The horizontal component of the pressure on a plate is equal to the pressure per square inch at its center of gravity multiplied by the area of its vertical projection, and the vertical component of the pressure is equal to the pressure at its center of gravity multiplied by its horizontal projection.* **(5)**

Examples. 1. What will be the horizontal and vertical components of the pressures on a plate, BC, as in Fig. 9, which is inclined at an angle of 10° to the vertical, the length l of the plate

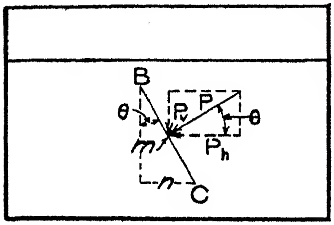


Fig. 9.

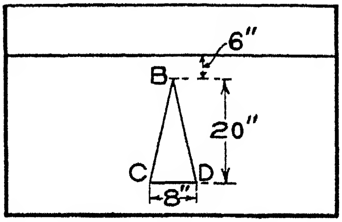


Fig. 10.

being 30 in. and the width 10 in., and the center being 2 feet below the water surface.

Here the pressure per sq. in. at the center is that due to a head of 2 ft., or is equal to $2 \times .434 = .868$ lb. per sq. in. The vertical projection of the plate is equal to $30 \times \cos 10°$ and its horizontal projection $= 30 \times \sin 10°$. Cos. $10° = .985$ and $\sin 10° = .174$, hence by rule 5 the required horizontal component $= .868 \times 30 \times .985 \times 10 = 256$ lb., and the vertical component $= .868 \times 30 \times .174 \times 10 = 45$ lb. Ans.

2. Required the horizontal and vertical components of the pressures on the three faces of the wedge shown in Fig. 10, the length of the wedge perpendicular to the paper being 12 in.

Face BC. The depth of the center of BC below the surface is evidently 16 in. The pressure per sq. in. at this depth $= \frac{16}{12} \times .434 = .579$ lb. The vertical projection of $BC = 20 \times 12$

= 240 sq. in., and its horizontal projection = $4 \times 12 = 48$ sq. in., whence the desired components are: Horizontal component = $240 \times .579 = 13.89$ lb., vertical component = $48 \times .579$ = 27.8 lb.

Face BD. The pressures are the same as on BC, the horizontal component acting towards the left and the vertical component acting downwards. The total downward pressure = 2×27.8 = 55.6 lb.

Face CD. The pressure per sq. in. at this depth = $.434 \times \frac{26}{12}$ = .940 lbs. Total upward pressure = $8 \times 12 \times .940 = 90.2$ lb.

9. Pressure on Curved Surfaces. If we are dealing with a curved surface as BC, Fig. 11, the pressure is still at all points normal to the surface, but the varying direction of the pressures makes it difficult to determine readily the resultant pressure. The results of the preceding article will, however, enable us to solve the problem sufficiently accurate for all purposes. Suppose the pressure on each square inch be resolved into vertical and horizontal components. Each of these components will equal the normal pressure at the center of the square inch multiplied by the horizontal or vertical projection of the inch of area. Adding all together we find that the total horizontal pressure will equal a certain average horizontal pressure multiplied by the vertical projection of the entire area, and the vertical pressure will equal a certain average vertical pressure multiplied by the horizontal projection. It can be shown that the average value of the horizontal pressure is equal to the pressure at the center of gravity of the vertical projection, but the average value of the vertical pressure cannot be readily determined with accuracy. It may always be estimated by taking as near as may be a pressure corresponding to the average depth of the area below the water surface. Where the body is submerged a great distance, or is under a great pressure in a closed vessel, the error will be unimportant.

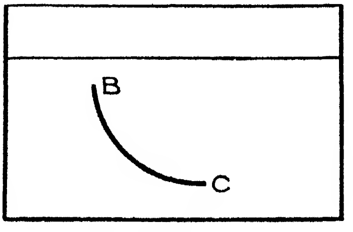


Fig. 11.

10. Bursting Pressure in Pipes and Cylinders. Let BECD, Fig. 12, be the cross-section of any pipe of diameter d and length l and containing water under a head h. The figure shows the pipe connected to an open vessel with water standing at a height h above the center. This free surface of water may represent a reservoir at a height h above the pipe, or the pipe may be entirely closed and the pressure head h exerted upon the water by means of a force pump or a pumping engine. The pressure per square inch at the center of the pipe will be $h \times .434$ pounds. The pressure against the pipe BEDC will be perpendicular to the surface at all points, and if the diameter is small compared to the height h, this pressure will be practically the same at all points and equal to $h \times .434$ pounds per square inch.

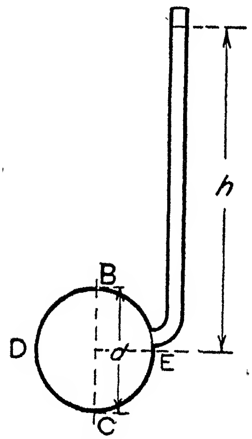


Fig. 12.

Suppose we wish to find the total horizontal force acting against the half BDC. By the foregoing article we may consider the pressure on its vertical projection BC. The center of gravity of this vertical projection will be at the center of the pipe and the pressure per square inch at that point will be $h \times .434$. The area of the projection BC is equal to $d \times l$. Hence the total horizontal pressure against BDC will equal $h \times .434 \times dl$. The pressure against the side BEC will be the same, but opposite in direction.

The action of the pressures on BDC and BEC tends to burst the pipe at points B and C. This is resisted by the stress in the pipe, the amount of which at each of these points is one-half the total horizontal pressure on BDC or BEC, or equal to $\frac{1}{2} h \times .434 \times dl$. If we consider a length of pipe of only one inch then $l = 1$ and we have the important formula for the bursting stress in a pipe:

$$S = \frac{1}{2} h \times .434 \times d \qquad \textbf{(6)}$$

in which $S =$ stress per lineal inch of pipe

h = head of water in feet

and d = diameter of pipe in inches.

By expressing the pressure-head in pounds per square inch instead of feet head we have

$$S = \frac{pd}{2} \qquad (7)$$

in which p = pressure per square inch at center of pipe.

If t = thickness of pipe in inches and s = stress on the metal per square inch then

$$s = \frac{pd}{2t} \qquad (8)$$

For large pipes and low heads the stress at C will be a little larger than at B.

11. Longitudinal Stress in Closed Pipes and Cylinders. Let Fig. 13 represent a side view of a short pipe or cylinder, closed at

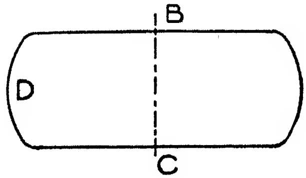


Fig. 13.

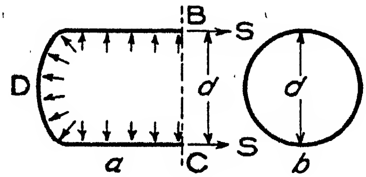


Fig. 14.

the ends like a steam boiler and containing water under a pressure p per square inch. Consider the portion to the left of a section BC. (See Fig. 14a.) The cross-section of the cylinder at BC will be a circle of diameter d on which there will be a stress S due to the horizontal water pressure on the end of the cylinder at D. This total horizontal pressure may be found as in the preceding article. It is equal to the average pressure p multiplied by the vertical projection of the area of the end. This projection, Fig. 14b, is equal to the area of the circle of diameter d, or to $\frac{1}{4}\pi d^2$. Hence the total horizontal force is $p \times \frac{1}{4}\pi d^2$, and hence

$$\text{Total stress} = p \times \tfrac{1}{4}\pi d^2.$$

This stress is distributed entirely around the circumference of the cylinder, or over a distance equal to πd. The stress per inch of circumference is then equal to

$$S' = \frac{p \times \frac{1}{4}\pi d^2}{\pi d} = \frac{pd}{4} \qquad (9)$$

This is seen to be just one-half of the stress in a circumferential direction, as given by formula 7.

If t equal thickness of cylinder, then the horizontal stress per square inch of metal is

$$s' = \frac{pd}{4t} \qquad (10)$$

Examples 1. What will be the stress per lineal inch in a pipe 30 in. in diameter under a water pressure of 40 feet?

The pressure per lineal inch is equal to, by equation 6, $\frac{1}{2} \times 40 \times .434 \times 30 = 260.4$ lb. Ans.

2. If the safe strength of the metal of a pipe in example 1 is 2,000 lb. per sq. in., what will be the necessary thickness of the pipe wall?

The stress per lineal inch is 260.4 lb., and if the safe stress is 2,000 lb. per sq. in., the necessary thickness will be equal to $260.4 \div 2{,}000 = .130$ inch. Ans.

EXAMPLES FOR PRACTICE.

1. What is the bursting stress per square inch in a pipe $\frac{1}{2}$ inch thick and 4 feet in diameter under a pressure head of 400 feet? 8,330 lb. Ans.

2. What is the stress per square inch in a boiler plate 1 inch thick, the boiler being 6 feet in diameter working under a pressure of 150 lb. per sq. in. (Use equation 8.) 5,400 lb. Ans.

3. What is the horizontal stress per sq. in. in the boiler of example 2? 2,700 lb. Ans.

12. Center of Pressure on Rectangular Areas. In the preceding discussion of Arts. 7 and 8 the *total* pressure was the quantity determined. In the case of the plate reaching to the surface, Fig, 6, the variation in the pressure was represented by a triangle of forces, and where the plate was wholly submerged, Fig. 7, it was represented by a trapezoid. In either case the "center of pressure," or the point where the resultant of the pressure forces would be applied, will be opposite the *center of gravity* of the

pressure area. In the case of the triangle the center of gravity is two-thirds the distance from the apex to the base, hence,

The center of pressure against a rectangular plate which reaches to the surface, or projects above it, is two-thirds the distance from the surface to the lower edge of the plate. (11)

In the case of the wholly submerged plate, with trapezoidal pressure diagram BC, Fig. 15, the pressure head at B is equal to h_1 and that at C h_2, which heads will be equal to the heights BD and CE of the trapezoid representing the pressures.

By the method explained in the paper on Strength of Materials, Art. 48, we find the center of gravity of the trapezoid of length l to be at a distance from B equal to

$$x = \frac{2}{3} l \frac{\frac{h_1}{2} + h_2}{h_1 + h_2} \qquad (12)$$

Another form of expression can be obtained by noting that if the point A be the intersection of the plane BC, produced, with the surface of the water, the line DE, produced, will also pass through A, since a plate AC would have a triangle of pressures

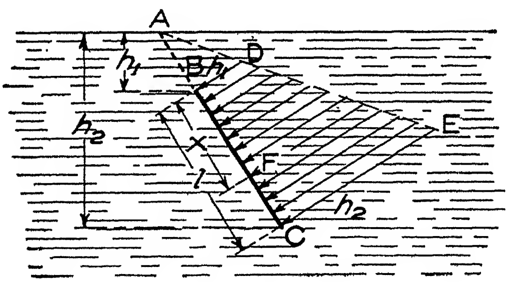


Fig. 15.

represented by AEC. Then by proportion we would have $\frac{h_1}{h_2} = \frac{AB}{AC}$, or $h_1 = h_2 \frac{AB}{AC}$. Substituting this value for h_1 in equation 12, and reducing, we have

$$x = \frac{2}{3} l \frac{\frac{AB}{2} + AC}{AB + AC} \tag{13}$$

13. Center of Pressure on Plane Areas of Any Form. The center of pressure of irregular plane areas can be found by the following rule, the demonstration of which is here omitted. Let BC, Fig. 15, represent a plane area of any form, then

The distance AF from the surface to the center of pressure is equal to the moment of inertia of the given area about an axis at A divided by the product of the area times the distance from A to its center of gravity. **(14)**

Examples. 1. What force S will be required to lift a sluice gate BC, Fig. 16, placed on the sloping face of a dam and hinged at B? The gate is 3 feet wide, 4 feet long from B to C, and has such a slope that the vertical projection BD = 3.5 ft., and the

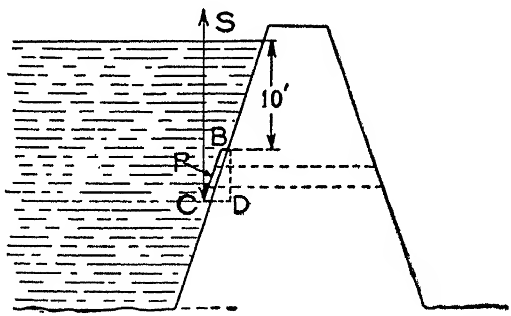


Fig. 16.

horizontal projection CD = 1.93 ft. The depth of B below the surface is 10 ft.

We will first find the total pressure P against the gate. By Art. 7 this will be the pressure per sq. in. at the center multiplied by the area. The depth of the center is $10 + \frac{1}{2}\,BD = 10 + \frac{3.5}{2} = 11.75$ feet. The pressure per sq. ft. $= 11.75 \times 62.5 = 734$ lb. The total pressure $= 734 \times 3 \times 4 = 8{,}808$ lb.

The center of pressure will be found next. This is at a distance from B given by formula 12, in which $h_1 = 10$ and $h_2 = 13.5$. We have then $x = \frac{2}{3} \times 4 \times \frac{\frac{10}{2} + 13.5}{10 + 13.5} = 2.1$ feet.

Now taking moments about B we have $S \times 1.93 = P \times 2.1$ or $S = 8,808 \times \frac{2.1}{1.93} = 9,590$ lb. Ans.

2. Find the water pressure on a gate AB, Fig. 17, one foot long, when the heads on the two sides are different; also find the reactions R_1 and R_2 of the gate against sills at A and B.

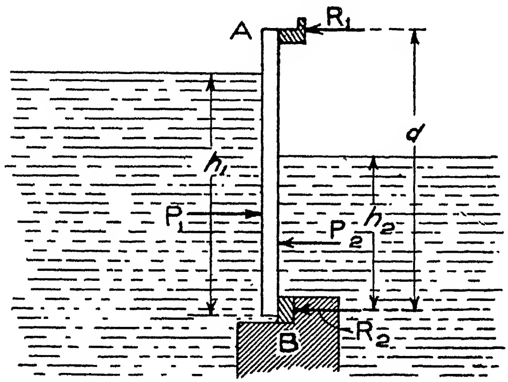


Fig. 17.

By Art. 7 the total pressure P_1 of the water on the left side of the gate is equal to the pressure at the half depth $\frac{h_1}{2}$ multiplied by its submerged area. Taking here the square foot as the unit and letting w = weight of a cubic foot of water, the pressure at a depth $\frac{h_1}{2}$ is $\frac{h_1}{2} \times w$ pounds per square foot, and as the exposed area is $h_1 \times 1$ the total pressure $P_1 = \frac{h_1}{2} \times w \times h_1 = \frac{1}{2} h^2_1 \times w$. The center of pressure, or point of application of P_1, is $\frac{2}{3} h_1$ below the surface (Art. 12).

In like manner the pressure $P_2 = \frac{1}{2} h^2_2\, w$, and its point of application is $\frac{2}{3} h_2$ below the water surface on that side.

The forces P_1 and P_2 being known, the reaction R_1 may be found by taking moments about B as explained in Mechanics.

There results the equation

$$R_1 \times d - P_1 \frac{h_1}{3} + P_2 \frac{h_2}{3} = 0$$

whence

$$R_1 = \frac{1}{3} \frac{P_1 h_1 + P_2 h_2}{d}$$

Substituting the values of P_1 and P_2 above given, we have

$$R_1 = \frac{1}{3} \times w \times \frac{1}{2} \frac{h^3_1 + h^3_2}{d}$$

$$= \frac{1}{6} w \frac{h^3_1 + h^3_2}{d} \qquad \textbf{(15)}$$

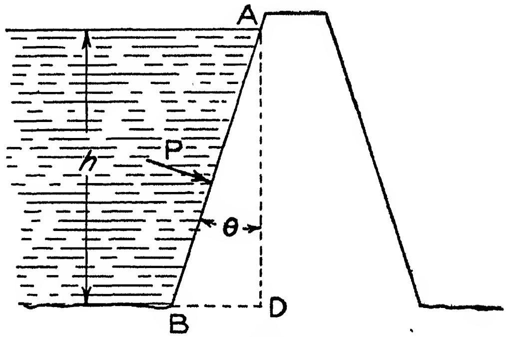


Fig. 18.

in which all dimensions are to be expressed in feet, and the result will be for a gate one foot long. For other lengths the value of R_1 will be proportional to the length.

3. Find the pressure P on a dam AB, Fig. 18. Let h = depth of water against the dam. Consider a length of dam of one foot. By Art. 7 the total pressure P is equal to the pressure at the half depth multiplied by the area of AB or

$$P = \frac{h}{2} \times w \times (\text{length of AB}) \times 1.$$

The center of pressure, by Art. 12, is two-thirds of the distance from A to B.

The horizontal component of the pressure is, by Art. 8, equal to the pressure per square foot at mid-depth multiplied by the vertical projection of the face AB, or

$$P_h = \frac{h}{2} \times w \times h \times 1 = \frac{1}{2} wh^2 \qquad (16)$$

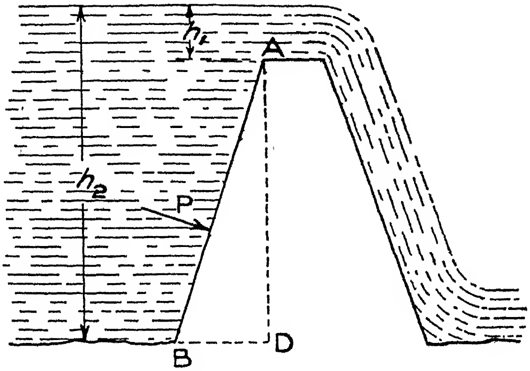


Fig. 19.

The vertical component is likewise

$$P_v = \frac{h}{2} \times w \times (\text{length BD}),$$

but we can write BD $= h \tan \theta$. Hence

$$P_v = \frac{1}{2} wh^2 \tan \theta. \qquad (17)$$

If the dam is submerged, as shown in Fig. 19, then the method employed in example 1 of this Article must be used.

EXAMPLES FOR PRACTICE.

1. What is the horizontal pressure on a dam one foot long on which the water has a depth of 80 feet; and where is the center of pressure? 200,000 lb., and 26 ft. 8 in. from the bottom. Ans.

2. What is the vertical component of the pressure in example 1 if the face of the dam slopes 1 inch horizontally to 1 foot vertically? (The horizontal projection $= \frac{1}{12} \times 80 = 6\frac{2}{3}$ ft.)

16,670 lb. Ans.

3. In Fig. 19 if $h_1 = 10$ ft., $h_2 = 40$ ft., AD $= 30$ ft., and BD $= 10$ ft., wh^t will be the horizontal and vertical components

of the pressure P? The center of gravity of the area is 30 ft. deep. Use rule 5.

Hor. comp. = 46,875 lb.; Vert. comp. = 15,625 lb. Ans.

4. How far from A is the center of pressure in example 3? The length of AB = 31.62 ft. Use equation (12). 18.97 ft. Ans.

14. Buoyant Effect of Water on Submerged Bodies. If a body AB, Fig 20, be submerged, the water exerts an uplift upon it owing to the fact that the pressure upwards on the bottom of the body is greater than the pressure downwards on the top. The net upward force, or buoyant effect, is exactly equal to the weight

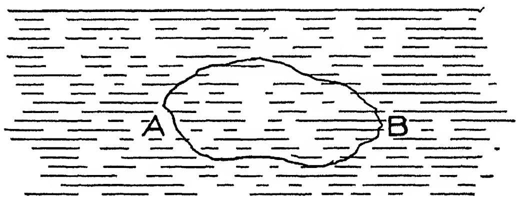


Fig. 20.

of a volume of water equal to that of the body AB. It is plain that this must be so, for if AB be replaced by water, the water would tend neither to rise nor fall, that is, it would be just supported by the surrounding pressures. Hence the following well-known law:

The weight of a body in water is less than its weight in air by an amount equal to the weight of an equal volume of water.

15. *The Specific Gravity* of a substance is the ratio of its weight to that of an equal volume of water. The specific gravity is found by weighing a body in air and then in water. The difference is the weight of an equal volume of water. Then if W equals weight in air; and W′ equals weight in water, then W - W′ = weight of water displaced, and

$$\text{Specific gravity} = \frac{W}{W - W'}, \qquad \textbf{(18)}$$

as explained in Elementary Mechanics.

EXAMPLES FOR PRACTICE.

1. If a body weighs 100 lb. in air and 40 lb. in water, what is its specific gravity? 1.67. Ans.

2. If a body of .6 cu. ft. in volume weighs 75 lb., what is its specific gravity, the weight of water being 62.5 lb. per cu. ft.? 2.0. Ans.

3. If a body of 3 cu. ft. in volume has a specific gravity of .75, what force is necessary to submerge it? Here the buoyant effect is greater than the weight of the body. 46.9 lb. Ans.

FLOW OF WATER THROUGH ORIFICES.

16. Velocity of Flow Through Orifices. If AB, Fig. 21, be a vessel containing water of depth h, and C and D are any open tubes connected therewith, the water will stand in these tubes at

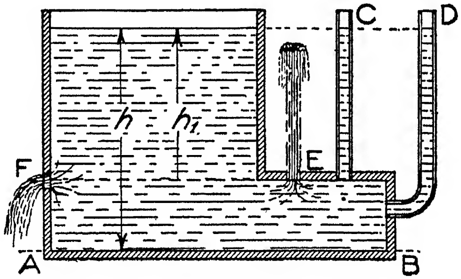


Fig. 21.

the same height h above the base level AB as in the large vessel, and the pressure in the tubes at any given depth is the same as in the large vessel at the same depth. If we now make an opening at E so that the water will issue in a vertical direction it has been experimentally demonstrated that the water will rise very nearly to the same level as it will in the tubes. The discrepancy is due to the air resistance and a slight friction at the opening. Neglecting this discrepancy the velocity of the water at E can be determined on the principle that it must be sufficient to cause the water to rise the distance h_1. In Mechanics it was shown that, neglecting air resistance, the velocity a body must have to cause it to rise against gravity a distance h_1 is the same as the velocity acquired by a body falling through the same distance. This velocity is given by formula

$$v = \sqrt{2gh}$$

in which v = velocity in feet per second

g = acceleration of gravity

= 32.2 feet per second

and h = height of fall, or the height a body will rise when started with a velocity v.

Applying this to the jet issuing from E we find that the theoretical velocity of efflux is

$$v = \sqrt{2gh_1}$$

If the orifice be in the side of the vessel, as at F, on the same level as E, it is plain that the water will issue with the same force as at E, since the pressure is the same. Hence in general:

The theoretical velocity of efflux from an orifice in any direction is

$$v = \sqrt{2gh} \qquad (19)$$

where h is the pressure head in feet at the orifice.

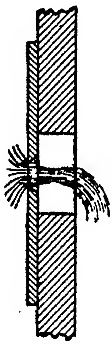


Fig. 22.

In practice the velocity is a little less than that given by the formula, the actual velocity being from 97% to 99% of the theoretical. This ratio of .97 to .99 is called the *coefficient of velocity*.

17. Use of Orifices for Measuring Water. In making use of an orifice for measuring water it is desirable, for the sake of accuracy, that the orifice be constructed in such a way that the water in passing out will touch the inner edge only. This may be done by making the orifice of a very thin plate, or cutting it on a bevel so that the water will not come in contact with the side, as shown in Fig. 22. To get accurate results an orifice should be made of metal, such as brass, and fastened to the inside of the tank as in Fig. 22, but in many cases sufficiently accurate results can be obtained by cutting a beveled hole in the side of a tank.

To give reliable results the orifice should be located a distance from the nearest side or the bottom of the tank not less than three times the width of the orifice. The tank or channel should also have a cross-section much larger than that of the orifice so

that the velocity of the water as it approaches the orifice will be small, otherwise the discharge will be affected by this "velocity of approach". If the cross-section of the tank is as much as twenty times that of the orifice this effect is of no consequence.

18. Discharge Through Small Orifices. When water flows through an orifice, such as shown in Fig. 22, the direction of the flow at the edges is such as to cause the water vein to contract as it issues from the orifice. The area of the contracted vein at its smallest section is only 60 to 70 per cent of the full area of the orifice, the exact value depending upon the size of the orifice, and the pressure. Now the discharge through any orifice, pipe, or channel is equal to the area of the cross-section of the stream of water multiplied by its velocity at that point. If we measure the cross-section in square feet and the velocity in feet per second, then the discharge will be expressed in cubic feet per second. In the case of the orifice, then, to determine the discharge per second we would need to multiply the area of the cross-section of the vein of water by its velocity. In Art. 16 it was shown that the actual velocity of the jet was about 97 to 99 per cent of the theoretical velocity v, which refers to the velocity of the vein at the contracted section where the velocity is a maximum. The discharge will then be found by multiplying this actual velocity by the actual area at the point of contraction. Thus if we take the coefficient of velocity as .98 and the coefficient of contraction as .65, the discharge would be

$$Q = .98\ v \times .65\ A$$

where Q = discharge in cubic feet per second

v = theoretical velocity in feet per second by equation 19, and A = area of orifice in square feet.

If we substitute for v its value $\sqrt{2gh}$ we have

$$Q = .98 \times .65\ A \sqrt{2gh}$$
$$= .637\ A \sqrt{2gh},$$

The coefficient .637 in this case is called the *coefficient of discharge*, and as it varies with different conditions, it is desirable to use the more general formula

$$Q = c\ A \sqrt{2gh}. \qquad (20)$$

in which c = coefficient of discharge, which varies in value from about .66 to .60. This coefficient is the product of the "coefficient of velocity" and the "coefficient of contraction."

TABLE NO. 3.

Coefficients for Circular Vertical Orifices.

Head, h, in Feet.	Diameter of Orifice in Feet.						
	0.02	0.04	0.07	0.10	0.2	0.6	1.0
0.4	....	0.637	0.624	0.618			
0.6	0.655	.630	.618	.613	0.601	0.593	
0.8	.648	.626	.615	.610	.601	.594	0.590
1.0	.744	.623	.612	.608	.600	.595	.591
1.5	.637	.618	.608	.605	.600	.596	.593
2.0	.632	.614	.607	.604	.599	.597	.595
2.5	.629	.612	.605	.603	.599	.598	.596
3	.627	.611	.604	.603	.599	.598	.597
4	.623	.609	.603	.602	.599	.597	.596
6	.618	.607	.602	.600	.598	.597	.596
8	.614	.605	.601	.600	.598	.596	.596
10	.611	.603	.599	.598	.597	.596	.595
20	.601	.599	.597	.596	.596	.596	.594
50	.596	.595	.594	.594	.594	.594	.593
100	.593	.592	.592	.592	.592	.592	.592

TABLE NO. 4.

Coefficients for Square Vertical Orifices.

Head, h, in Feet.	Side of the Square in Feet.						
	0.02	0.04	0.07	0.1	0.2	0.6	1 0
0.4	....	0.643	0.628	0.621			
0.6	0.660	.636	.623	.617	0.605	0.598	
0.8	.652	.631	.620	.615	.605	.600	0.597
1.0	.648	.628	.618	.613	.605	.601	.599
1.5	.641	.622	.614	.610	.605	.602	.601
2.0	.637	.619	.612	.608	.605	.604	.602
2.5	.634	.617	.610	.607	.605	.604	.602
3	.632	616	.609	.607	.605	.604	.603
4	.628	.614	.608	.606	.605	.603	.602
6	.623	.612	.607	.605	.604	.603	.602
8	.619	.610	.606	.605	.604	.603	.602
10	.616	.608	.605	.604	.603	.602	.601
20	.606	.604	.602	.602	.602	.601	.600
50	.602	.601	.601	.600	.600	.599	.599
100	.599	.598	.598	.598	.598	.598	.598

TABLE NO. 5.

Coefficients for Rectangular Orifices 1 Foot Wide.

Head, h, in Feet.	Depth of Orifice in Feet.						
	.125	.25	.50	.75	1.0	1.5	2.0
.4	.634	.633	.622				
.6	.633	.633	.619	.614			
.8	.633	.633	.618	.612	.608		
1.	.632	.632	.618	.612	.606	.626	
1.5	.630	.631	.618	.611	.605	.626	.628
2.	.629	.630	.617	.611	.605	.624	.630
2.5	.628	.628	.616	.611	.605	.616	.627
3.	.627	.627	.615	.610	.605	.614	.619
4.	.624	.624	.614	.609	.605	.612	.616
6.	.615	.615	.609	.604	.602	.606	.610
8.	.609	.607	.603	.602	.601	.602	.604
10.	.606	.603	.601	.601	.601	.601	.602
20.				.601	.601	.601	.602

19. Experimental Coefficients of Discharge. Many experiments have been made on different kinds of orifices to determine the value of c, equation 20, so that by means of this formula and a table of coefficients, orifices could readily be used for measuring water. The accompanying tables give these coefficients for circular, square, and rectangular orifices in vertical planes, the rectangular orifices all being one foot wide.

Example 1. What is the discharge from a circular orifice 3 in. in diameter under a pressure head of 10 feet?

By Table No. 3 the coefficient of discharge for an orifice of a diameter of .25 ft. and under a head of 10 feet is .597. The area of the orifice $= \frac{1}{4} \times 3.14 \times .25^2 = .049$ sq. ft. Then by equation 20 the discharge will be $.597 \times .049 \times \sqrt{2 \times 32.2 \times 10} = .743$ cu. ft. per sec. Ans.

2. What will be the velocity of flow in example 1, the coefficient of velocity being taken equal to .97?

By equation 19 the velocity $= .97 \sqrt{2 \times 32.2 \times 10} = 24.6$ ft. per sec. Ans.

EXAMPLES FOR PRACTICE.

1. What will be the discharge from an orifice 4 in. square under a head of 16 feet? 2.14 cu. ft. per sec. Ans.

2. What must be the diameter of a circular orifice acting under a head of 25 feet to discharge 1 cu. ft. per sec.? (Assume $c = .6$ for a trial solution.) 2.76 in. Ans.

3. A pipe discharges 1.5 cu. ft. per sec. into a tank from which the water escapes through an orifice 6 in. square. How

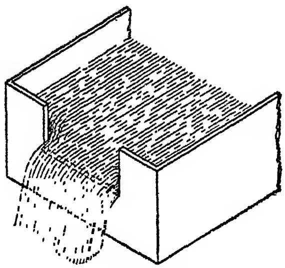


Fig. 23*a*.

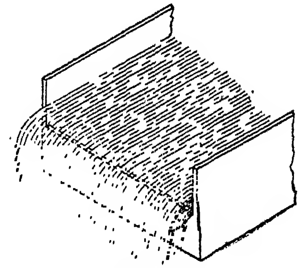


Fig. 23*b*.

deep will the tank be filled above the orifice when the outflow is just equal to the inflow? 1.53 ft. Ans.

FLOW OF WATER OVER WEIRS.

20. General Explanation. The term weir is usually given to a notch cut in the side of a tank or reservoir through which water may flow and be measured. The notch is usually rectangular and may have a width less than that of the tank, as shown in Fig. 23*a*, or equal to that of the tank, as in Fig. 23*b*. Such weirs are often used for measuring the flow of a small stream by building a small dam and leading all the water through a notched plank or timber wall. For accurate work weirs should be sharp-crested (the "crest" is the lower edge over which the water passes) so that the water will touch the inner corner only as in the case of the standard orifice described in Art. 18. The back side of the weir should be smooth and vertical for a considerable distance downwards from the crest.

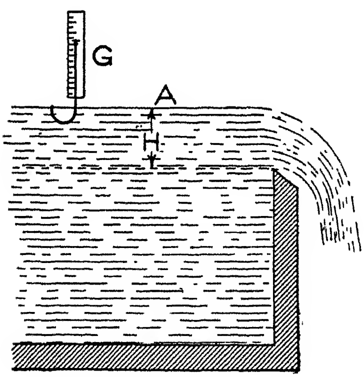


Fig. 24.

If the weir is made as in Fig. 23a the water in passing out will cause a contraction of the stream laterally, but if made as in Fig. 23b the water will pass out parallel to the sides of the tank and there will be no lateral, or, as it is called, "end contraction". In either case reliable results may be obtained by the use of the proper coefficients, but if the form of Fig. 23a be used, the distance of the notch from the side of the tank or channel should be at least three times the depth of the water on the weir in order that the contraction may be complete.

The measurement of water flowing over a weir is accomplished by merely measuring the depth of the water flowing over it. Then knowing this and the length of the weir the discharge can be calculated. In measuring this depth of water, or "height" of water on the weir, as it is commonly called, it is necessary to take the level of the water some distance back from the weir, as at A, Fig. 24, in order to avoid the effect of the curvature of the water surface. The difference between the level of the water and that of the weir is then the desired height H. The necessary distance back from the weir may be taken as 2 or 3 feet for small weirs to 8 or 10 feet for large ones.

A common and accurate way of determining the level of the water at A is by means of a submerged hook, shown at G, Fig. 24, called a *hook gauge*, arranged to be easily moved vertically along a scale. Fig. 25 shows such a gauge in detail. The gauge is set by moving it until the hook comes to the surface of the water. The scale is then read and the level of the water determined.

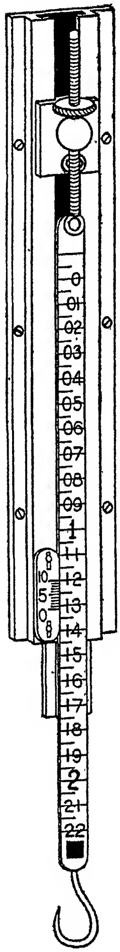


Fig. 25.

21. Formulas for Discharge. If the weir were a rectangular orifice at a considerable depth below the surface its discharge would be given by the formula

$$Q = c \times b \times d \sqrt{2gh} \qquad \textbf{(21)}$$

as in equation 20 of Art. 18. In this expression b = breadth and d = height of orifice, and h = average depth of orifice below the

surface, or the average pressure head. In the case of the weir the depth d is the height H, and the average pressure head, h, is something less than H, varying from nothing for the water at the surface to the full value H for the water at the crest. For a case like this where the square root of a quantity, h, is taken, that varies from zero to a given value H, the *average* value of this square root is two-thirds the square root of the maximum limit H. That is, for h in equation 20 we may substitute $\frac{2}{3}\sqrt{H}$, giving for the discharge

$$Q = cb \times \frac{2}{3} H \sqrt{2gH}$$

$$= c \times \frac{2}{3} b \sqrt{2g} \times H^{\frac{3}{2}} \qquad \textbf{(22)}$$

in which c is the coefficient of discharge and equal to about .60 to .65 as for orifices.

If the channel is small the "velocity of approach" will have an appreciable effect upon the discharge, increasing it somewhat above what it otherwise would be. This is taken account of by calculating approximately the velocity of the water in the channel of approach at the place where the level of the water is measured, and determining the head h corresponding to this velocity by the formula

$$h = \frac{v^2}{2g}.$$

Then the discharge will be, for weirs with end contractions,

$$Q = c \times \frac{2}{3} b \sqrt{2g}\,(H + 1.4h)^{\frac{3}{2}} \qquad \textbf{(23)}$$

and for weirs without end contractions

$$Q = c \times \frac{2}{3} \sqrt{2g}\ \left(H + 1\frac{1}{3} h\right)^{\frac{3}{2}} \qquad \textbf{(24)}$$

The coefficient c should, in all cases, be selected according to the character of the weir.

In calculating "velocity of approach", it is necessary first to get an approximate value for the discharge Q by omitting the term h. The resulting discharge, divided by the cross-section of the

tank or channel will be, with sufficient accuracy, the desired velocity of approach.

22. Coefficients of Discharge. Tables Nos. 6 and 7 give values of the coefficient c for the above formulas for rectangular sharp-crested weirs.

TABLE NO. 6.

Coefficients for Contracted Weirs.

Effective Head in Feet, h.	Length of Weir in Feet, b.						
	0.66	1	2	3	5	10	19
0.1	0.632	0.639	0.646	0.652	0.653	0.655	0.656
0.15	.619	.625	.634	.638	.640	.641	.642
0.2	.611	.618	.626	.630	.631	.633	.634
0.25	.605	.612	.621	.624	.626	.628	.629
0.3	.601	.608	.616	.619	.621	.624	.625
0.4	.595	.601	.609	.613	.615	.618	.620
0.5	.590	.596	.605	.608	.611	.615	.617
0.6	.587	.593	.601	.605	.608	.613	.615
0.7	....	.590	.598	.603	.606	.612	.614
0.8	....	....	.595	.600	.604	.611	.613
0.9	....	....	.592	.598	.603	.609	.612
1.0	....	....	.590	.595	.601	.608	.611
1.2	....	....	.585	.591	.597	.605	.610
1.4	....	....	.580	.587	.594	.602	.609
1.6	....	....	....	.582	.591	.600	.607

TABLE NO. 7.

Coefficients for Weirs without Contractions.

Effective Head in Feet, h.	Length of Weir in Feet, b.						
	19	10	7	5	4	3	2
0.1	0.657	0.658	0.658	0.659			
0.15	.643	.644	.645	.645	0.647	0.649	0.652
0.2	.635	.637	.637	.638	.641	.642	.645
0.25	.630	.632	.633	.634	.636	.638	.641
0.3	.626	.628	.629	.631	.633	.636	.639
0.4	.621	.623	.625	.628	630	.633	.636
0.5	.619	.621	.624	.627	.630	.633	.637
0.6	.618	.620	.623	.627	.630	.634	.638
0.7	.618	.620	.624	.628	.631	.635	.640
0.8	.618	.621	.625	.629	.633	.637	.643
0.9	.619	.622	.627	.631	.635	.639	.645
1.0	.619	.624	.628	.633	.637	.641	.648
1.2	.620	.626	.632	.636	.641	.646	
1.4	.622	.629	.634	.640	.644		
1.6	.623	.631	.637	.642	.647		

23. The Francis Formula. The most widely used weir formula for large weirs without end contractions is that derived by Mr. James B. Francis from an extensive series of experiments on weirs 10 feet long. His formula is

$$Q = 3.33\, b H^{\frac{3}{2}}, \qquad (25)$$

in which the unit of length must be the foot. This is equivalent to the use of a constant value of the coefficient c of equation 22, equal to .623. It gives results sufficiently close for most purposes. With end contractions the length b is to be reduced by .1 H for

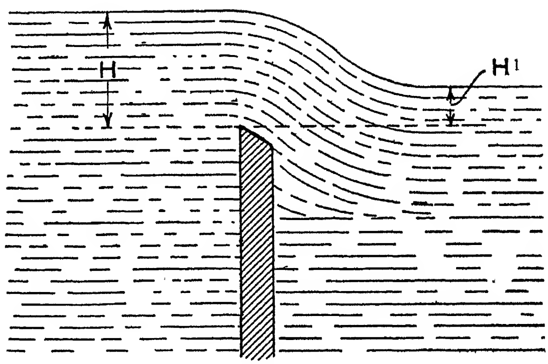


Fig. 26.

one end contracted and by .2 H for both ends contracted. The formula is further modified to allow for velocity of approach, but where this element enters, use may be made of the other formula.

24. Submerged Weirs. Where the water on the down-stream side of a weir is higher than the crest, as in Fig. 26, the discharge is closely given by the formula

$$Q = 3.33\, b\, (n\mathrm{H})^{\frac{3}{2}}, \qquad (26)$$

where H is the height of the water on the upper side and n is a coefficient depending on the ratio of the head on the lower side, H^1, to the head H. The values of n are as follows:

TABLE NO. 8.

Values of n for Submerged Weirs.

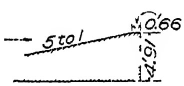
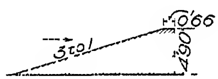
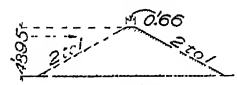
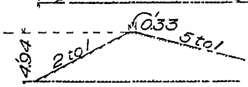
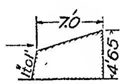
$\frac{H'}{H}$	n	$\frac{H'}{H}$	n	$\frac{H'}{H}$	n	$\frac{H'}{H}$	n
.00	1.000	.20	0.985	.45	0.912	.70	0.787
.02	1.006	.25	0.973	.50	0.892	.75	0.750
.05	1.007	.30	0.959	.55	0.871	.80	0.703
.10	1.005	.35	0.944	.60	0.846	.90	0.574
.15	0.996	.40	0.929	.65	0.819	1.00	0.000

25. Weirs of Irregular Section. In many cases it is desirable to determine the flow of a stream by measurements taken of the height of water flowing over some dam or weir; and, on the other hand, in the design of waste-weirs some method of estimating their capacity is essential. The law of flow over such weirs

TABLE NO. 9.

Values of the Coefficient C in the Formula

$$Q = CbH^{\frac{3}{2}} \text{ for irregular weirs.}$$

	Form of Weir.	Height on Weir in Feet.							
		1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
1		3.51	3.37	3.33	3.31	3.29	3.23	3.16	3.14
2		....	3.76	3.68	3.68	3.70	3.75	3.83	
3		.....	3.68	3.71	3.81	3.90	4.00	4.06	
4		3.81	3.61	3.68	3.65	3.72	3.80	3.93	
5		3.81	3.61	3.57	3.63	3.62	3.67	3.71	3.80

is very complicated, and the only accurate way of determining the constants for any particular case is by means of experiments on a section of the same form as the one in question. If this is impos-

sible, the best substitute for it is to use constants which have been determined for a weir agreeing as closely in form as may be to the one under consideration.

In Table No. 9 are given several sets of coefficients for five forms of dams, as determined by experiment. This coefficient is to be used in place of the value 3.33 in equation 25.

It will be noted by comparing Nos. 1 and 3 that the discharge falls off considerably by using a flat slope for the back of the dam.

Examples. 1. What will be the discharge of a sharp-crested weir 4 ft. long with H = 6 inches, there being contraction at both ends?

By Table No. 6 the coefficient may be taken at .610. Then by equation 22, $Q = .61 \times \frac{2}{3} \times 4 \times \sqrt{64.4} \times \left(\frac{1}{2}\right)^{\frac{3}{2}} = 4.6$ cu. ft. per sec. Ans.

2. If the channel of approach in example 1 be 6 feet wide by $2\frac{1}{2}$ feet deep, what will be the effect of the "velocity of approach"?

Assuming the same discharge as above, the velocity of flow in this channel will be $\frac{4.6}{6 \times 2\frac{1}{2}} = .11$ ft. per sec. The head h corresponding to this velocity $= \frac{v^2}{2g} = \frac{.11^2}{64.4} = .0002$ ft. Introducing this value for h in equation 23, it is seen that the additional term $1.4\ h$ is too small to be of any practical consequence.

FLOW OF WATER THROUGH PIPES.

26. Discharge Through Pipes for Different Velocities. The rate of discharge through a pipe is equal to the average velocity of the flowing water multiplied by the cross-section of pipe. Velocities are usually expressed in feet per second and discharge in cubic feet per second or gallons per minute. The diameter of a pipe is always given in inches. These differences in units make it desirable to have a table at hand giving for a velocity of one foot per second the discharge of pipes of various diameters expressed both in cubic feet per second and in gallons per minute. Such a table is given below:

TABLE NO. 10.

Discharge of Pipes in Cubic Feet Per Second and in Gallons Per Minute for a Velocity of One Foot Per Second.

(For other velocities multiply the discharge here given by the velocity expressed in feet per second.)

Diameter of Pipe in Inches.	Discharge.	
	Cubic Feet Per Second.	Gallons Per Minute.
1	.0055	2.4
2	.0218	9.8
3	.0491	22.0
4	.0873	39.1
6	.1964	88.1
8	.3491	157
10	5454	245
12	.7854	352
14	1.069	480
16	1.396	627
20	2.182	978
24	3.142	1410
30	4.909	2200
36	7.069	3155
42	9.621	4317
48	12.568	5639

EXAMPLES FOR PRACTICE.

1. What will be the discharge in gallons per minute of a 6-inch pipe for a velocity of 4.5 feet per sec.? 396 gallons per min. Ans.

2. What velocity will be required to discharge 1,000,000 gallons per day through an 8-inch pipe? 4.4 ft. per sec. Ans.

3. What diameter of pipe will be required to discharge 1,000 gals. per min. at a velocity of 5 feet per sec.?

An 8-inch pipe will discharge 785 gals. per min. and a 10-inch pipe will discharge 1,225 gals. per min. A 10-inch pipe would therefore be necessary if no intermediate size is available. Ans.

27. General Principles Governing the Flow of Water Through Pipes. Let ABCD, Fig. 27, be any pipe leading from a reservoir and having a stop valve at D. Also suppose B*b* and C*c* are tubes connected with the pipe at B and C and open at the top.

From the laws of pressure explained in Art. 6 we know that if the valve D be closed so that there will be no motion of the water the water will rise in the tubes Bb and Cc to the same level as that in the reservoir. The pressure at A will be represented by the head h_1 and that at B and C by the heads h_2 and h_3 respectively, which heads may be greater or less than the head at A according as the pipe slopes downwards or upwards from A.

Now let the valve at D be opened partly so as to permit the water to escape slowly. It will be found that the pressures at B and C will immediately decrease and that the water in the tubes will fall to some lower levels b' and c'. This decrease in pressure

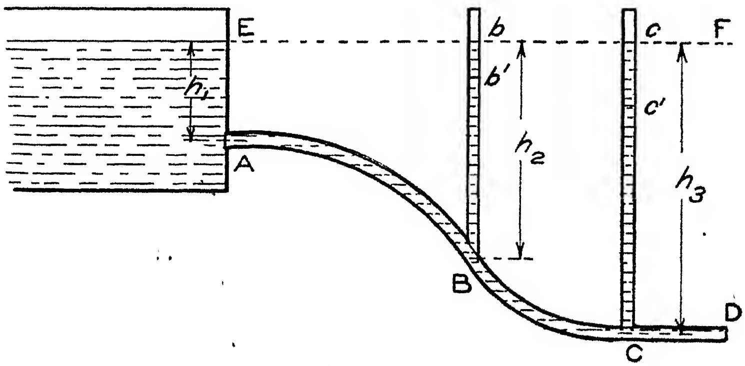


Fig. 27.

is due to two causes. First, a part of the pressure head has been used to give the water some velocity in the pipe, and second, a part has been consumed in the friction of the water in passing from A to B and C. The portion used in giving the water its velocity is the same as the head required to produce a given velocity of efflux, v, from an orifice, and is found from the formula $v = \sqrt{2gh}$. Solving for h we have

$$h = \frac{v^2}{2g} \qquad (27)$$

in which v is the actual velocity of flow in the pipe.

The pressure head lost in friction is usually much greater than that used in velocity and is the most important as well as the most difficult part of the problem of determining the flow in pipes.

If H represents the total loss of pressure head between the reservoir and any point B, h_v the head necessary to produce the given velocity at B ("velocity head") and h_f the pressure lost by friction between A and B we then have in general

$$H = h_v + h_f \qquad (28)$$

or from equation 27

$$H = \frac{v^2}{2g} + h_f \qquad (29)$$

In the figure, bb' represents the head H for point B and cc' that for point C. Between B and C the loss in head is the *difference* between bb' and cc' and is all due to friction, since the velocity is the same at the two points, the pipe being of uniform size.

If now we open the valve D farther so as to give the water a higher velocity the level of the water in the tubes bB and cC will fall still more, that is, there will be a greater loss of pressure head, H, than before. This increase in loss of pressure is due mainly to the increased friction loss h_f caused by the higher velocity, but to a small extent also to the increased energy transformed into velocity head.

In any case that part of the head H needed to produce the velocity v, which is equal to $\frac{v^2}{2g}$, can readily be calculated or can be obtained from the following table:

TABLE NO. 11.

Velocity Heads

$$h = \frac{v^2}{2g}$$

Corresponding to Various Values of v.

v feet per sec.	h ft.	v ft. per sec.	h ft.	v ft. per sec.	h ft.	v ft. per sec.	h ft.
2.0	0.06	4.0	0.25	6.0	0.56	8.0	0.99
2.2	0.08	4.2	0.28	6.2	0.60	8.2	1.04
2.4	0.09	4.4	0.30	6.4	0.64	8.4	1.10
2.6	0.10	4.6	0.33	6.6	0.68	8.6	1.15
2.8	0.12	4.8	0.36	6.8	0.72	8.8	1 20
3.0	0.14	5.0	0.39	7.0	0.76	9.0	1.26
3.2	0.16	5.2	0.42	7.2	0.80	9.2	1.31
3.4	0.18	5.4	0.45	7.4	0.85	9.4	1.37
3 6	0.20	5.6	0.49	7.6	0.90	9.6	1.43
3.8	0.22	5.8	0.52	7.8	0.94	9.8	1.49

The usual problem in practice consists in calculating the friction loss h_f between any two given points in a pipe for a given velocity v; or, conversely, to determine the velocity which will occur with a given loss of head h_f.

28. Formulas for Friction Loss in Pipes. A great number of experiments have been made to determine the friction loss in the flow of water through pipes. The results show great variations due to many causes, chief of which is the variation in the character of the pipe as to material, degree of roughness of the interior, diameter, etc. Consequently much less accuracy is possible in the estimation of the flow of water through pipes than through orifices or over weirs. Theory is of very little assistance here, and the only practicable method of calculation is to express by some formula the approximate law of variation in friction, and then use coefficients as determined from experiments.

Results of experiments show that the friction loss in a pipe is approximately proportional to the length of the pipe and to the square of the velocity of the water, and is inversely proportional to the cross-section of the pipe divided by its circumference. If we let

h_f = loss by friction between any two points;
l = length of pipe between same two points;
v = velocity of water in pipe;
r = ratio of cross-section to circumference of pipe, called the "hydraulic mean radius",

we then have, according to the above law,

$$h_f = \frac{v^2 l}{r} \times k$$

where k is some coefficient.

It is usual to write this formula so as to express directly the value of v. By solving for v we have

$$v = \sqrt{r\frac{h_f}{l}} \times \frac{1}{\sqrt{k}}$$

Putting C for $\frac{1}{\sqrt{k}}$, we may write

$$v = C\sqrt{r\frac{h}{l}} \qquad (30)$$

which is known as the Chezy formula. The values of r, h, and l are to be expressed in feet, and the result will give v in feet per second.

The above formula may be used for all kinds of pipe by using a suitable value of C as determined by experiments on similar pipe. For ordinary cast iron pipe the value of C varies from about 100 for pipes 1 or 2 inches in diameter to 140 or 150 for pipes 4 or 5 feet in diameter. Various diagrams and formulas for C have been devised for cast iron pipe, all of which are more or less unsatisfactory. Mr. Hamilton Smith has constructed a diagram which is probably as satisfactory as any now in use. This diagram is not entirely convenient in form, and instead of it we give below an extended table giving the actual velocities of flow v for various diameters of pipe and various losses of head for a length of 100 feet for pipes from $\frac{3}{4}$ in. to 3 in. in diameter, and for a length of 1,000 feet for larger pipes. This table is very convenient to use in calculations, as the desired velocity or loss of head can be seen at a glance.

TABLE NO. 12.

Discharge, Friction Head, and Velocity of Flow Through Smooth Pipes such as Cast Iron.

Discharge, Gals. per Minute.	¾-inch Pipe.		1-inch Pipe.		1½-inch Pipe.	
	Loss of Head, Feet per 100 Feet.	Velocity, Feet per Second.	Loss of Head, Feet per 100 Feet.	Velocity, Feet per Second.	Loss of Head, Feet per 100 Feet.	Velocity, Feet per Second.
1	0.5	0.72	0.02	0.41		
2	2.0	1.4	0.6	0.82		
3	4.0	2.2	1.1	1.2		
4	7.2	2.9	1.8	1.6		
5	11.0	3.6	2.6	2.0		
6	15.0	4.3	3.6	2.4		
7	20.4	5.1	4.8	2.9		
8	25.5	5.8	6.2	3.3		
9	32.0	6.5	7.7	3.7		
10	39.0	7.2	9.4	4.1	1.1	1.8
12			13.0	4.9	1.6	2.2
14			17.1	5.7	2.2	2.5
16			21.8	6.5	2.8	2.9
18			27.1	7.3	3.5	3.3
20			33.0	8.2	4.3	3.6
30					9.5	5.4
40					16.0	7.2
50					24.0	9.1
60					34.0	10.9
70					45.0	12.7

TABLE NO. 12.—Continued.

Dicharge, Gals. per Minute.	2-inch Pipe.		2½-inch Pipe.		3-inch Pipe.	
	Loss of Head, Feet per 100 Feet.	Velocity, Feet per Second.	Loss of Head, Feet per 100 Feet.	Velocity, Feet per Second.	Loss of Head, Feet per 100 Feet.	Velocity, Feet per Second.
10	.4	1.0	0.1	0.65	0.05	0.45
20	1.2	2.0	0.4	1.3	0.2	0.90
30	2.4	3.1	0.8	1.9	0.4	1.4
40	4.0	4.1	1.4	2.6	0.7	1.8
50	6.1	5.1	2.1	3.3	1.0	2.3
60	8.6	6.1	2.9	3.9	1.4	2.7
70	11.5	7.1	3.9	4.6	1.8	3.2
80	14.8	8.2	5.0	5.2	2.3	3.6
90	18.4	9.2	6.3	5.9	2.8	4.1
100	22.2	10.2	7.7	6.5	3.4	4.5
120			10.8	7.8	4.8	5.4
140			14.3	9.1	6.3	6.3
160			18.3	10.4	8.0	7.2
180			22.7	11.8	9.9	8.1
200			27.5	13.1	12.0	9.0
250					18.0	11.3
300					25.0	13.6

TABLE NO. 12.—Continued.

Discharge, Gals. per Minute.	4-inch Pipe.		6-inch Pipe.		8-inch Pipe.	
	Loss of Head, Feet per 1,000 Feet.	Velocity, Feet per Second.	Loss of Head, Feet per 1,000 Feet.	Velocity, Feet per Second.	Loss of Head, Feet per 1,000 Feet.	Velocity, Feet per Second.
50	2.3	1.3				
75	5.2	1.9				
100	8.7	2.5	1.2	1.1		
125	13.1	3.2	1.8	1.4		
150	18.3	3.8	2.5	1.7		
175	24.3	4.5	3.3	2.0		
200	31.0	5.1	4.2	2.3	1.1	1.3
250	46.5	6.4	6.3	2.8	1.6	1.6
300	65.0	7.7	8.9	3.4	2.2	1.9
350			11.9	4.0	2.9	2.2
400			15.1	4.5	3.7	2.6
450			18.7	5.1	4.6	3.9
500			22.7	5.7	5.6	3.2
600			31.8	6.8	7.9	3.8
700			42.2	7.9	10.5	4.5
800			54.0	9.1	13.4	5.1
900					16.6	5.8
1000					20.2	6.4
1100					24.1	7.0

TABLE NO 12.—Continued.

Discharge, Gals. per Minute.	10-inch Pipe. Loss of Head, Feet per 1,000 Feet.	10-inch Pipe. Velocity, Feet per Second.	12-inch Pipe. Loss of Head, Feet per 1,000 Feet.	12-inch Pipe. Velocity, Feet per Second.	16-inch Pipe. Loss of Head, Feet per 1,000 Feet.	16-inch Pipe. Velocity, Feet per Second.
200	.35	.82	.14	.57		
300	.73	1.2	.30	.85		
400	1.24	1.6	.51	1.1		
500	1.87	2 0	.78	1.4	.18	.80
600	2.6	2.4	1.10	1.7	.26	.96
700	3.5	2.9	1.45	2.0	.34	1.1
800	4.4	3.3	1.82	2.3	.43	1.3
900	5.5	3.7	2.3	2.6	.54	1.4
1000	6.7	4.1	2.8	2.8	.66	1.6
1100	8.0	4.5	3.3	3.1	.78	1.8
1200	9.4	4.9	3.9	3.4	.92	1.9
1300	10.9	5.3	4.5	3.7	1.06	2.1
1400	12.6	5.7	5.1	4.0	1.22	2.2
1500			5.8	4.2	1.38	2.4
1600			6.5	4.5	1.55	2.6
1700			7.3	4.8	1.74	2.7
1800			8.1	5.1	1.93	2.9
1900			9.0	5.4	2.1	3.0
2000			9.9	5.7	2.3	3.2
2200			11.7	6.2	2.8	3.5
2400					3.3	3.8
2600					3.8	4.2
2800					4.4	4.5
3000					5.0	4.8
3500					6.6	5.6

Examples. 1. What is the head lost in friction due to the flow of 800 gallons per minute in a 6-inch pipe?

From Table No. 12 we see that the friction head in a 6-inch pipe for a flow of 800 gals. per min. is 54.0 ft. for each 1,000 ft. of pipe. Ans.

2. What size of pipe will be required to convey 700 gallons of water per minute a distance of 8,000 feet with a total loss of head of 40 feet?

The loss of head per 1,000 ft. is $40 \div 8 = 5$ ft. From the table we find that for a discharge of 700 gallons per min. the loss of head in an 8-in. pipe is 10.5 ft. per 1,000, and in a 10-in. pipe it is 3.5 ft. A 10-in. pipe would then be required if the assumed loss is not to be exceeded. Ans.

3. If a town is supplied with water from an elevated reservoir through a pipe line 15,000 feet long, how high must the

TABLE NO. 12.—Continued.

Discharge, Gals. per Minute.	20-inch Pipe.		24 inch Pipe.		30 inch Pipe.	
	Loss of Head, Feet per 1,000 Feet.	Velocity, Feet per Second.	Loss of Head, Feet per 1,000 Feet.	Velocity, Feet per Second.	Loss of Head, Feet per 1,000 Feet.	Velocity, Feet per Second.
1000	.23	1.0	.08	.71		
1200	.32	1.2	.12	.85		
1400	.42	1.4	.16	.99		
1600	.52	1.6	.20	1.1		
1800	.64	1.8	.25	1.3		
2000	.77	2.0	.31	1.4	.10	.91
2200	.92	2.2	.37	1.6	.12	1.00
2400	1.08	2.5	.43	1.7	.14	1.09
2600	1.25	2.7	.50	1.8	.17	1.18
2800	1.43	2.9	.58	2.0	.19	1.27
3000	1.62	3.1	.66	2.1	.22	1.36
3200	1.82	3.3	.74	2.3	.24	1.45
3400	2.04	3.5	.83	2.4	.27	1.55
3600	2.27	3.7	.92	2.5	.30	1.64
3800	2.51	3.9	1.02	2.7	.33	1.73
4000	2.76	4.1	1.12	2.8	.36	1.82
4500	3.43	4.6	1.39	3.2	.46	2.05
5000	4.16	5.1	1.68	3.5	.56	2.27
5500	4.96	5.6	2.00	3.9	.67	2.50
6000	5.80	6.1	2.35	4.3	.78	2.73
6500					.90	2.96
7000					1.03	3.18
7500					1.17	3.41
8000					1.32	3.64
9000					1.64	4.09
10000					2.00	4.55

reservoir be above the town and what must be the size of the pipe line so that the pressure of water in the distributing pipes be not less than 60 pounds per sq. in., equivalent to $60 \times 2.3 = 138$ ft. head. The amount of water required is 1,800 gals. per min.

This problem has several solutions since various sizes of pipe may be assumed and the reservoir placed at the elevation to furnish the necessary pressure. An examination of Table No. 12 shows that to deliver 1,800 gals. per min. a 12-in. pipe would consume in friction 8.1 ft. of head per 1,000 ft., a 16-in. pipe would consume only 1.93 ft. per 1,000 ft., and a 20-in. pipe only .64 ft. No value is given for a 10-in. pipe, but it would evidently be 20 feet or more per 1,000, which would give a total loss for 15,000 ft. of 300 feet, a loss which would ordinarily be impracticable.

If we use a 12-in. pipe the total loss in friction will be 8.1 × 15 = 121.5 ft. The velocity of flow will be 5.1 ft. per sec. and the necessary velocity head, h_v, will be, by Table No. 11, .4 ft. The total head = 121.5 + .4 = 121.9 ft., and the necessary elevation of the reservoir = 138 + 121.9 = 259.9 ft. above the town.

If a 16-inch pipe be assumed, the friction loss = 1.93 × 15 = 28.9 ft., the velocity head = .1 ft., and the total head = 29 ft. Elevation of reservoir = 29.0 + 138 = 167 ft.

If a 20-inch pipe be used, the friction head = .64 × 15 = 9.6 ft., the velocity head is less than .1 ft. and may be neglected. The required height of reservoir = 147.6 ft.

Still larger sizes will give still lower elevations for the reservoir, but it is evident that the reservoir in any case must have an elevation somewhat greater than 138 ft.

From the above results we see that a 12-in. pipe requires the reservoir to be at an elevation of 259.9 ft., a 16-in. pipe requires an elevation of 167 ft., and a 20-in. pipe an elevation of 147.6 ft. The proper size to use would be that size which would give the cheaper construction for the pipe and reservoir combined.

29. The Hydraulic Grade Line. Referring again to Fig. 27, it will be seen that the drop in pressure between B and C will be proportional to the length of the pipe from B to C, and if we have a long pipe with several open tubes attached to it like *b*B and *c*C, the level of the water in them would be lower and lower as we proceed along the pipe, the drop being uniform so long as the pipe is of the same size and kind, the amount of the drop per 1,000 feet being given in Table No. 12. If now a line were drawn from E through the points *b*, *c*, etc., so that the height of this line above the pipe would represent the pressures in it, this line would be called the "hydraulic grade line" for the pipe under the given conditions. It is convenient in various problems to construct such a grade line. Its position will evidently vary with the velocity of the flow and will be a horizontal line when the water is still, and always a straight line for a pipe of uniform conditions.

30. Siphons. If in any case a pipe line rise above this hydraulic grade line, as shown in Fig. 28, the pressure in such portion of the pipe will be less than atmospheric, the pressure measured by the grade line as described above referring in all cases to the pres-

sure in excess of the usual atmospheric pressure. That portion of the pipe BC lying above the grade line is called a *siphon*. The greatest height above the grade line which it is practicable to operate a siphon is considerably less than the height of the water barometer given in Art. 4. Evidently since the velocity of flow, and hence the hydraulic grade line, can be varied by varying the opening at D, a pipe which may act as a siphon at one time may not so act at another. Thus in the figure, if valve D be nearly closed so that the flow is reduced and hence also the frictional loss,

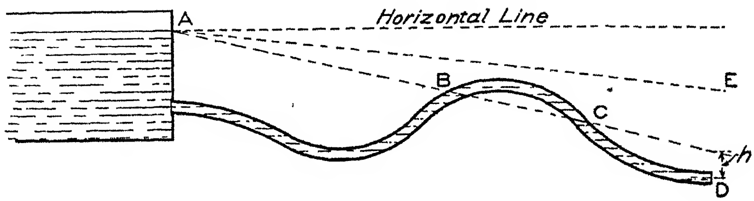


Fig. 28.

the grade line will rise to some position such as AE and there will be pressure in excess of atmospheric at all points.

31. Flow Through Special Forms of Pipes. *Riveted Pipe.* The friction loss in riveted pipes depends upon the thickness of the plates and the manner of making the joints. Experiments on this class of pipes are not sufficiently numerous to enable any general expression to be formulated, so that in the design of such pipes the selection of coefficients must be made by reference to the experimental data. In general it is found that the coefficient C, of equation 30, changes little with change in diameter or velocity, and in this respect exhibits considerable difference from its variation in cast-iron pipe. For ordinary velocities the value of C, for new pipe appears to range between 100 and 115. A value of 100 is as great as it is well to use.

32. Wood Stave Pipe. Few experiments have been made on this class of pipe although it has been used quite extensively in the West. The pipe is usually quite smooth and not subject to deterioration on the interior, so that its discharging capacity is high. For ordinary velocities the value of C, equation 30, may be taken at 110.

33. Fire Hose. In making provisions for fire protection it becomes necessary to estimate the effectiveness of a stream of water when led through a given length of hose for a given pressure at the hydrant, or to find what pressure is required to throw a stream a given height or a given distance. The usual size of fire hose is $2\frac{1}{2}$ inches. At the end of the hose is attached a nozzle of a diameter usually of 1 in., $1\frac{1}{8}$ in., or $1\frac{1}{4}$ in., which partly controls the amount and pressure of the water discharged. If there were no friction in the hose the water could be thrown nearly to a height corresponding to the pressure head at the hydrant, but the hose friction is very great, and two or three hundred feet of hose will cut down the effective pressure often more than one-half. Evidently the more rapid the flow through the hose the greater the friction loss, hence if the nozzle is small so that the discharge will be small, the effective pressure near the nozzle will be greater than with a large nozzle and large discharge. Hence a higher stream can be thrown through a small nozzle with a given hydrant pressure and length of hose than through a large nozzle, although the stream is not so effective in quenching a fire as the larger stream.

In Table No. 13 are given the necessary data for estimating the loss of head and effectiveness of fire streams for various pressures and for three sizes of nozzles

In the table, page 44, the pressure given is that at the nozzle instead of at the hydrant. To get the latter, it is necessary to add to the nozzle pressure the head lost in the hose. The result will be the hydrant pressure, providing nozzle and hydrant are at same level. If not, then a correction would need to be made for this difference in elevation. The vertical height and horizontal distances are to be measured from the nozzle. The heads are given in pounds per square inch, which is the customary unit in this class of work. To reduce to feet of head multiply pounds pressure by 2.3.

Examples. 1. What hydrant pressure will be required to throw a stream of water 75 feet vertically through a $1\frac{1}{8}$-in. nozzle and 300 feet of hose.

In the table for the $1\frac{1}{8}$-in. nozzle we see that for a height of 75 feet the loss of head per 100 feet of hose is 20 pounds, and the pressure at the nozzle is (in first column of table) 50 pounds. The

TABLE NO. 13.

Hose and Fire-Stream Data.

Pressure at Nozzle (Base of Play-pipe).	1-inch Smooth Nozzle.					$1\frac{1}{8}$-inch Smooth Nozzle.					$1\frac{1}{4}$-inch Smooth Nozzle.				
	Discharge in Gallons per Minute.	Loss of Head per 100 Feet of Ordinary Hose.	Vertical Height of Jet for Good Fire-streams.	Maximum Horizontal Distance for Good Fire-streams.	Extreme Drops at Level of Nozzle.	Discharge in Gallons per Minute.	Loss of Head per 100 Feet of Ordinary Hose.	Vertical Height of Jet for Good Fire-streams.	Maximum Horizontal Distance for Good Fire-streams.	Extreme Drops at Level of Nozzle.	Discharge in Gallons per Minute.	Loss of Head per 100 Feet of Ordinary Hose	Vertical Height of Jet for Good Fire-streams.	Maximum Horizontal Distance for Good Fire-streams.	Extreme Drops at Level of Nozzle.
lb.		lb.	ft.	ft.	ft.		lb.	ft.	ft.	ft.		lb	ft.	ft.	ft.
20	132	5	35	37	77	168	8	36	38	80	209	12	37	40	83
30	161	7	51	47	109	206	12	52	50	115	256	19	53	54	119
40	186	10	64	55	133	238	16	65	59	142	296	25	67	63	148
50	208	12	73	61	152	266	20	75	66	162	331	31	77	70	169
60	228	15	79	67	167	291	24	83	72	178	363	37	85	76	186
70	246	17	85	72	179	314	28	88	77	191	392	43	91	81	200
80	263	20	89	76	189	336	32	92	81	203	119	49	95	85	213
90	279	22	92	80	197	356	36	96	85	214	444	55	99	90	225
100	295	25	96	83	205	376	40	99	89	224	468	62	101	93	236

hydrant pressure will then be $50 + (20 \times 3) = 110$ pounds per square inch. The discharge will be about 266 gallons per minute.

2. With a hydrant pressure of 100 pounds, what will be the discharge through 250 feet of hose with a $1\frac{1}{4}$-in. nozzle, and how high can such a stream be thrown with effectiveness?

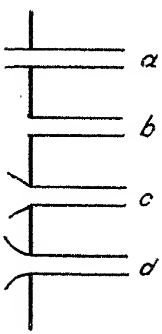


Fig. 29.

This problem must be solved by trial. In the table for $1\frac{1}{4}$-in. nozzles, we see that for a discharge of 269 gallons the nozzle pressure is 40 pounds, and the loss of head per 100 feet of hose is 25 pounds; for a discharge of 331 gallons the nozzle pressure is 50 pounds, and the loss of head per 100 feet is 31 pounds, etc. We have given the head of 100 pounds, which must equal the sum of the nozzle pressure and the loss in the hose. If we try the first value for discharge, we have a nozzle pressure of 40 pounds and a total loss in the hose of $25 \times 2.5 = 62.5$ pounds, or a total of $40 + 62.5 = 102.5$ pounds. This being a little more than the total available head, it is evident that we have assumed too high a

discharge. The next lower value is 256 gallons, giving a nozzle pressure of 30 pounds and a total hose loss of 19 × 2.5 or 47.5 pounds, giving a total of 30 + 47.5 = 77.5 pounds. Evidently the correct value is somewhere between 296 and 256, and further that it is but very little below the former value. For a total change in discharge of 40 gallons we have a change in total head of 102.5 – 77.5 or 25 pounds. Hence for a change of 2.5 pounds the discharge will vary about $\frac{1}{10}$ of 40 gallons, or 4 gallons. The discharge may then be taken as 292 gallons per minute. The effective height will be between 67 feet and 53 feet, but only a little less than the former value, say 65 feet. This is as close an estimate as the conditions of the problem will warrant, since the hose friction is a factor that varies greatly according to the character of the hose.

34. Minor Losses of Head in Pipes. In most of the following formulas the quantity $\frac{v^2}{2g}$ occurs. For given values of v this quantity can readily be taken from Table No. 11.

Loss of Head at Entrance. This is expressed by the formula

$$h = \left(\frac{1}{c^2} - 1\right)\frac{v^2}{2g}, \qquad (31)$$

where v = velocity in the pipe, and c is the coefficient of discharge. For various forms at entrance, as shown in Fig. 29, we have the following values:

	c.	$\frac{1}{c^2} - 1$.
Pipe projecting into reservoir, Fig. (a)	.72	.93
End of pipe flush with reservoir, Fig. (b)	.82	.49
Conical or bell-shaped mouth, Fig. (c) or (d)	.93 to .98	.15 to .04

Loss of Head at Bends. For 90° bends this is equal to

$$h = n\frac{v^2}{2g} \qquad (32)$$

in which n has the following values according to the ratio of the radius of the pipe r to the radius of curvature R:

$\frac{r}{R}$	.1	.2	.3	.4	.5	.6	.7	.8	.9	1.0
n	.13	.14	.16	.21	.29	.44	.66	.98	1.41	1.98

Loss of Head in Valves. Weisbach's experiments on small gate-valves gave values for n in the expression $h = n\frac{v^2}{2g}$ as follows:

Ratio of height of opening to diameter.	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{8}$
Values of n	.07	.26	.81	2.1	5.5	17	98

In applying the above formula v is the velocity in the pipe.

For a throttle-valve placed at various angles θ with the axis of the pipe, Weisbach found the following values of n:

θ..	5°	10	20°	30°	40°	50°	60°	65°	70°
n...	.24	.52	1.5	3.9	11	33	118	256	750

Experiments on large gate-valves have been made by Kuichling and by J. W. Smith. The following table gives values of the coefficient c in the expression $Q = cA\sqrt{2gh}$. In this expression A is the area of the opening, h is the head lost in the valve, Q is the rate of discharge.

TABLE NO. 14.

Coefficients for Large Gate-Valves.

Ratio of height of opening to diameter	.05	.1	.2	.3	.4	.5	.6	.7	.8
Ratio of area of opening to total area	.05	.10	.23	.36	.48	.60	.71	.81	.89
Coefficient c for 24-in. valve	1.7	1.0	.72	.70	.77	.92	1.2	1.6	
Coefficient c for 30-in. valve	1.2	.9	.83	.82	.84	.90	1.05	1.35	2.1

Example. If a pump draws water from a pipe projecting into a reservoir what will be the loss of head at entrance, the velocity of water in the pipe being 6 feet per second

Using equation 31 of Art. 34 the value of $(\frac{1}{c^2} - 1)$ is, for this case, about .93. The loss of head is then $.93 \times \frac{v^2}{2g}$ which by Table No. 11 $= .93 \times .56$ or .52 feet. Ans.

If the pipe is flush with the reservoir the loss of head will be only $.49 \times .56$, or .27 feet.

Finally, if the pipe is enlarged to a bell-mouth or conical form the loss of head will be very small, say $.10 \times .56$ or .056 feet.

FLOW OF WATER IN OPEN CHANNELS.

35. General Formula. Where water flows in an open channel like a ditch, or a concrete, brick or tile sewer flowing less than full, the inclination of such channel is what furnishes the necessary fall or head to the water for overcoming friction. In this case there is no pressure at any point, and the loss of head from point to point will be the difference in level of the water surface between the given points. This difference in level, or head, after the flow has become steady is equal to the loss of head due to friction in the same distance.

The frictional loss in open channels is expressed by the same general formula as that used for pipes in Art. 28. It is

$$v = c\sqrt{rs} \tag{33}$$

in which as before

v = velocity in feet per second,
c = a coefficient,
r = hydraulic mean radius = the cross-section of the actual stream of water divided by that part of the perimeter that is under water ("wetted perimeter").
s = slope of channel, or ratio of fall to length $= \frac{h}{l}$.

For open channels the value of c varies much more than for pipes, as the nature of the channel varies more. Thus the channel may be a smooth tile sewer where c may be 100 or more, which is about the same as for iron pipe; or the channel may be a rough natural water-course for which the value of c will be only 30 or 40. Estimates of flow in very rough channels are obviously subject to great uncertainties, but for sewers and open masonry drains or conduits, estimates may be quite closely made, as the values of c have been quite well determined.

For convenience the value of c has been expressed in a formula, called Kutter's formula, in which the condition of the channel is taken account of by a special coefficient n, called the coefficient of roughness. This formula for ordinary cases is

$$c = \frac{\frac{1.8}{n} + 45}{1 + \frac{45n}{\sqrt{r}}} \qquad (34)$$

in which r = hydraulic mean radius in feet, and n = coefficient of roughness, varying from a value of about .009 for smooth plank to .030 for natural channels full of stone, etc.

The following are the values of n usually assumed for the various surfaces mentioned:

	n
Channels of well-planed timber	.009
" " neat cement or of very smooth pipe	.010
" " unplaned timber or ordinary pipe	.012
" " smooth ashlar masonry or brickwork	.013
" " ordinary brickwork	.015
" " rubble masonry	.017
" in earth free from obstructions	.020 to .025
" with detritus or aquatic plants	.030

After selecting the value of n, the value of c can readily be obtained from Table No. 15.

TABLE NO. 15.

Values of c in Kutter's Formula, for Various Values of n.

r in Feet.	Values of n.									
	.009	.010	.011	.012	.013	.015	.017	.020	.025	.030
.1	108	94	82	73	65	53	45	35	26	20
.2	129	113	100	89	80	66	56	45	31	26
.3	142	124	111	99	90	75	63	52	38	30
.4	150	132	118	106	96	80	69	56	42	34
.5	157	139	124	111	101	85	73	60	45	36
.6	162	143	128	116	105	89	76	63	48	38
.7	166	147	132	119	109	92	79	65	50	40
.8	170	151	135	122	112	95	82	68	52	42
.9	173	154	138	125	114	97	84	70	54	43
1.0	175	156	140	127	116	99	86	71	55	45
1.2	180	160	145	131	120	103	89	74	58	47
1.4	184	164	148	135	124	106	92	77	60	49
1.6	187	167	151	137	126	108	94	79	62	51
1.8	189	169	153	140	129	110	97	81	64	53
2.0	191	172	155	142	130	112	98	83	65	54
2.5	196	176	160	146	135	116	02	86	69	57
3.0	199	179	163	149	138	119	105	89	71	59
3.5	202	182	166	152	140	122	107	91	73	61
4.0	204	184	168	154	143	124	110	93	75	63
4.5	206	186	170	156	144	126	111	95	77	64
5.0	208	188	172	158	146	127	113	97	78	66

36. The Hydraulic Mean Radius r. As before explained, this is a name given to the quotient found by dividing the actual cross-section of a stream of water by the "wetted perimeter," or that part of the perimeter of the cross-section of the channel that is under water. In the case of a pipe flowing full, of diameter d, the cross-section is $\frac{1}{4}\pi d^2$ and the perimeter is πd, hence the value of r is $\frac{1}{4}\pi d^2 \div \pi d = \frac{1}{4}d$. For a pipe flowing half full it is, similarly, $\frac{1}{8}\pi d^2 \div \frac{1}{2}\pi d$ or $\frac{1}{4}d$, the same as when flowing full. When less than half full the cross-section of the stream falls off more rapidly than the wetted perimeter, so that the value of r decreases. Hence we see from equation 33 that the velocity also falls off.

For any given form of channel filled to a given point the value of r can readily be found by plotting the cross-section to a large scale and measuring the area and the wetted perimeter.

Example. What will be the velocity and discharge of water flowing in a concrete channel 4 ft. wide and 3 ft. deep and having a slope of 1 ft. per 1,000 ft.?

Equation 33 must be used. We will first get the values of r and s. The value of r is equal to the cross-section of the stream of water divided by the wetted perimeter $= \dfrac{3 \times 4}{4 + 3 + 3} = 1.2$ ft. The slope $s = \dfrac{1}{1,000} = .001$. The value of c is to be obtained from Table No. 15, n being taken at .013, say, the same as for brickwork. For $n = .013$ and $r = 1.2$ Table No. 15 gives $c = 120$. Substituting then in equation 33 we have $v = 120 \times \sqrt{1.2 \times .001} = 4.16$ ft. per sec. The discharge will be $4.16 \times 4 \times 3 = 49.92$ cu. ft. per sec. Ans.

37. Flow Through Ordinary Sewers. Sewers are usually constructed of vitrified earthen pipe or of brick or concrete. For the former material the value of n in equation 34 is usually taken at .013, and for brick and concrete about .015. If the concrete is smoothly finished n may be taken at .013.

The following Table No. 16 gives the velocities and discharges for circular sewers flowing full. For sewers flowing half full the velocity will be the same and the discharge one-half of the given values.

TABLE NO. 16.

Velocity and Discharge for Pipe Sewers (n = .013); Velocity in Feet per Second (V); Discharge in Cubic Feet Per Second (Q).

(For n = .011 add 20 per cent.)
(For n = .015 subtract 16 per cent.)

Fall of Sewer, in Feet per 100 ft.	4-inch		6-inch.		8-inch.		10-inch.		12-inch.		15-inch.		18-inch.	
	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q
10.	5.75	.50	7.99	1.57	10.04	3.50	11.94	6 51	13.73	10.78	16.24	19.93	18.59	32.86
5.	4.06	.35	5.64	1.11	7.09	2.48	8.43	4.60	9.70	7.62	11.48	14.08	13.13	23.22
4.	3.63	.32	5.05	.99	6.34	2.21	7.54	4.11	8.65	6.80	10.26	12.59	11.74	20.73
3.	3.15	.27	4.25	.83	5.49	1.92	6.53	3.56	7.51	5.90	8.89	10.91	10.17	17.97
2.	2.57	.22	3.56	.70	4.48	1.56	5.33	2 91	6.13	4.82	7.25	8.90	8.30	14.67
1.	1.82	.16	2.52	.49	3.17	1.11	3 77	2.06	4.33	3.40	5.13	6.30	5.87	10.38
.8	1.61	.14	2.25	.44	2.83	.99	3.37	1.84	3.87	3.04	4.59	5.63	5.25	9 28
.6	1.38	.12	1.95	.38	2.45	.86	2.92	1.59	3.35	2.64	3.97	4.89	4.55	8.01
.4			1 59	.31	2.00	.69	2.38	1.30	2.74	2.15	3.24	3.97	3.70	6.55
.2					1.40	.49	1.67	.91	1.91	1.51	2.27	2.79	2.60	4.60
.1							1.17	.64	1.35	1.06	1.60	1.96	1.83	3.24
.09											1.51	1.86	1.73	3 06
.08													1.63	2 88
.07													1.52	2.69

TABLE NO. 16.—Continued.

Fall of Sewer, in Feet per 100 ft.	20-inch.		22-inch.		24-inch.		30-inch.		33-inch.		36-inch,	
	V	Q	V	Q	V	Q	V	O	V	Q	V	Q
10.	20.08	43.8	21.51	56.8	22.91	72.0	26.84	131.7	28.69	170.3	30.46	215.3
5.	14.18	30.9	15.20	40.1	16.19	50.9	18.97	93.1	20.27	120.4	21.54	152.3
4.	12.69	27.7	13.59	35.9	14.47	45.5	16.96	83.3	18.13	107.7	19.26	136.5
3.	10.98	24.0	11.77	31.1	12.53	39.4	14.69	72.1	15.70	93.6	16.68	118.0
2.	8.97	19.6	9.61	25.4	10.23	32.2	11.99	58.9	12.82	76.1	13.62	96.3
1.	6.34	13.8	6.79	17.9	7.24	23.3	8.48	41.6	9.06	53.8	9.63	68.1
.8	5 67	12.4	6.07	16.0	6.47	20.3	7.58	37.2	8.11	48.1	8.61	60.9
.6	4.91	10.7	5.26	13.9	5.60	19.6	6.57	32.2	7.02	41.7	7.46	52.7
.4	4.00	8.7	4.29	11.3	4.56	14.3	5.35	26.3	5.72	34.0	6.08	43.0
.2	2.81	6.1	3.01	7.9	3.21	10.1	3.76	18.5	4.02	23.9	4.28	30.2
.1	1.98	4.3	2.12	5.6	2.26	7.1	2.66	13.0	2.84	16.9	3.02	21.3
.09	1.87	4.1	2.01	5.3	2.14	6.7	2.51	12.3	2.69	16.0	2.86	20.2
.08	1.76	3.8	1.89	5.0	2.02	6.3	2.37	11.6	2.53	15.0	2.69	19.0
.07	1.64	3.6	1.76	4.6	1.88	5.9	2.20	10.8	2.36	14.0	2.51	17.7
.06	1.51	3.3	1.63	4.3	1.73	5.4	2.04	10.0	2.18	12.9	2.32	16.4
.05			1.48	3.9	1.58	5.0	1.86	9.1	1.99	11.8	2.11	14.9
.04			1.32	3.5	1.40	4.4	1.65	8.1	1.77	10.5	1.88	13.3
.03					1.20	3.8	1.40	6.9	1.52	9.C	1.62	11.4
02					0.96	3.1	1.13	5.6	1.22	7.2	1.30	9.2

TABLE NO. 17.

Velocity and Discharge for Brick and Concrete Sewers ($n = .015$); Velocity in Feet per Second (V); Discharge in Cubic Feet Per Second (Q).

(For n = .013 add 19 per cent.)
(For n = .017 subtract 13 per cent.)

Fall of Sewer, in Feet per 100 ft.	33-inch.		36-inch.		42-inch.		4-foot.	
	V	Q	V	Q	V	Q	V	Q
.5	17.17	102.0	18.27	129.2	20.37	196 1	22.36	281 1
.4	15.36	91.2	16.34	115.5	18.21	175 3	20 00	251.3
.3	13.30	79.0	14.15	100 0	15.77	151.8	13.31	217.6
.2	10.85	64.5	11.55	81.7	12.88	123 9	14.13	177.6
.1	7.68	45.6	8.16	57.7	8 90	87 6	9.99	125.6
.8	6.86	40.7	7.30	51.6	8.14	78.3	8 93	112.3
.6	5.94	35.2	6.32	44.6	7 04	67.8	7 73	97.2
.4	4.84	28.8	5.15	36.4	5 75	54.0	6 31	79.3
.2	3.41	20.3	3.63	25.7	4.05	39.0	4.45	55.9
.1	2.40	14.3	2.52	18.1	2.85	27.5	3 13	39 4
.09	2.27	13.5	2.42	17.1	2.70	26 0	2 97	37 3
.08	2.14	12.7	2.28	16.1	2.55	24 5	2.80	35.2
.07	2.00	11.9	2.13	15.0	2 38	22 9	2 61	32 9
.06	1.85	11 0	1.97	13.9	2.20	21.1	2.42	30.4
.05	1.68	10 0	1.79	12.6	1.95	18.8	2 20	27.6
04	1.49	8.9	1.59	11.3	1.78	17 1	1 96	24 6
.03	1.28	7 6	1.37	9.7	1 53	14 7	1 68	21.2
.02					1.23	11.9	1.36	17 1
.15							1 16	14 6

TABLE NO. 17.—Continued.

Fall of Sewer, in Feet per 100 ft.	5-foot.		6-foot.		8-foot.		10-foot.	
	V	Q	V	Q	V	Q	V	Q
5.	26.05	512						
4.	23.30	457	26.34	745				
3.	20.17	396	22.81	645				
2.	16.47	323	18.62	527	22.53	1133	26.03	2045
1.	11.64	228	13.17	372	15.93	801	18 41	1446
.6	10.41	204	11.78	333	14.25	716	16.46	1293
.8	9.01	177	10.19	288	12.33	620	14.25	1119
.4	7.36	144	8.32	235	10.07	506	11.63	914
.2	5.19	102	5.87	166	7.10	357	8.21	645
.1	3.66	72	4.14	117	5.02	252	5.81	456
.09	3.47	68	3.92	111	4.76	239	5.51	433
.08	3.27	64	3.70	105	4.49	226	5 19	408
.07	3.05	60	3.46	98	4.20	211	4.86	382
.06	2 82	55	3.20	90	3 88	195	4.50	353
.05	2.57	50	2.92	82	3.54	178	4 10	322
.04	2.29	45	2.60	74	3.16	159	3.66	288
.03	1.97	39	2.24	63	2.73	137	3.17	249
.02	1.60	31	1 82	51	2.22	112	2.58	203
.015	1.37	27	1.56	44	1.93	97	2.23	175
.012			1.39	39	1.70	86	1.99	156
.010					1.55	78	1 81	143
.0095					1.25	63	1.77	139
.0090							1.72	135

MEASUREMENT OF THE FLOW OF STREAMS.

38. General Methods. For measuring the flow of a small stream the best method is by the use of a weir constructed of plank and built into a temporary dam of earth. Such weirs can readily be used for streams up to 3 or 4 feet in depth and 40 or 50 feet wide, although streams normally of such size would have flood flows many times greater and which could not be so measured. Where a dam already exists in a stream, observations of the flow over such a dam will give fairly good results when the coefficient of discharge is carefully selected as noted in Art. 25.

Where a weir cannot be used, then the flow must be measured by actually determining the mean velocity of the flow at a given

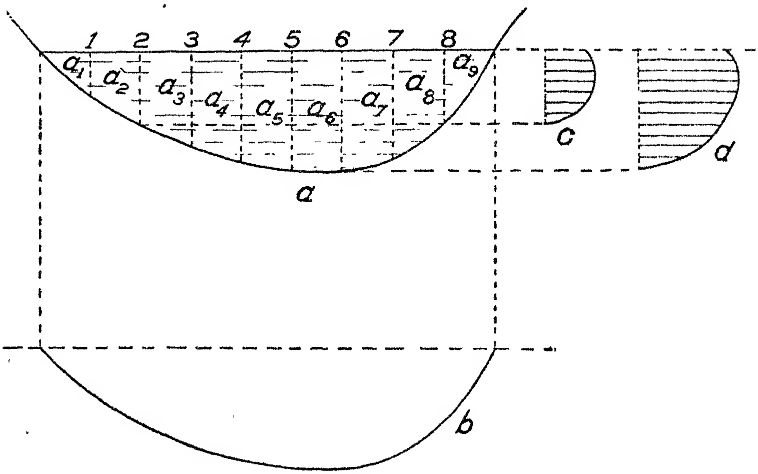


Fig. 30.

section and the area of such cross-section, then the discharge will be equal to the product of these quantities.

39. Variations in Velocity. Owing to the disturbing effect of the bottom and sides of a channel, the velocity of the water will not be the same at all points in a given cross-section. In general the velocity will be greater near the center of a stream than near the edges, and will be greater where the water is deep than where it is shallow. Thus if Fig. 30*a* represents the cross-section of a stream, the velocity of flow along the surface will vary in some such way as is represented in Fig. *b*, being greatest near the deep-

est parts and very small near the banks. Likewise if we consider the velocities along the vertical section 2 they will vary somewhat as shown in Fig. *e*, and at section 6 they will be as shown in Fig. *d*. In both Figs. *d* and *e* the maximum velocity is shown to be a little below the surface. This is usually the case, although it depends somewhat on the effect of the wind.

From these statements it will be seen that there are great variations in the velocity throughout the cross-section, and therefore the determination of the average velocity is not readily accomplished.

Instead of trying to get the average velocity through the entire cross-section, it is usual to divide the section of the stream into several vertical strips as shown in Fig. *a*. Then get the average velocity and discharge of each strip separately. In doing this a place should be selected where the flow is as uniform and the channel as regular as possible. In case floats are used to get velocities, as described later, it is necessary to establish two sections 100 feet apart or more, between which points the velocities are measured. In either case careful soundings must be taken and an accurate plot made of the cross-section, and the area of each division a_1, a_2, etc., determined. The divisions of the section may be marked by knots or tags on a rope stretched across the channel. The sections having been divided off, it remains to determine the average velocity in each.

40. Use of the Current Meter. The most accurate method of finding the velocity is by means of the current meter, one form of which is illustrated in Fig. 31.

The essential part of the current meter consists in the series of cups mounted on a wheel with vertical axis shown at the left of the vertical rod. This wheel being submerged, is rotated by the current, and the number of revolutions is recorded by an electrical device which may be held in a boat or on shore. The long vane attached to the wheel is to keep the meter always parallel with the current. A heavy weight is attached to the bottom of the rod to keep the meter steady, the whole apparatus being suspended by means of a rope from a boat or bridge. The number of revolutions per minute of the wheel being known, the velocity of the water at the wheel is calculated by multiplying by a coefficient determined by previous experiments with the meter.

The average velocity for any given strip is determined either by getting the velocity along the center of the strip at several different depths and taking the average, or by moving the meter slowly from top to bottom and then back to the top and taking a single reading. Whichever way determined the resulting velocity multiplied by the area of the strip in question equals the discharge of that strip. Then the total discharge equals the sum of the discharges of all the strips.

The coefficient to use in calculating actual water velocities

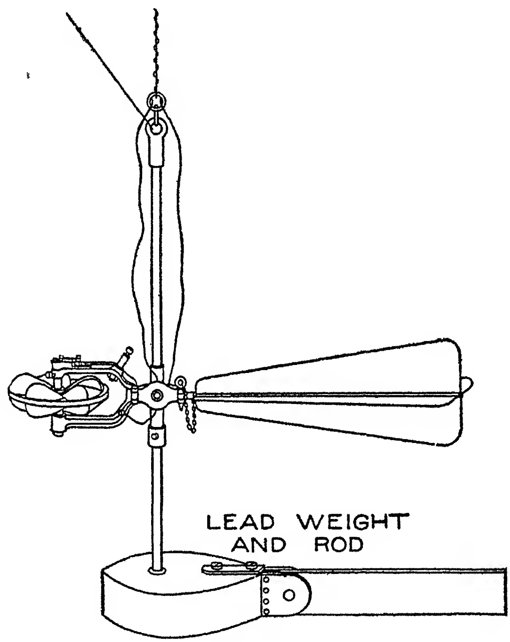


Fig. 31.

from meter readings is determined by a "rating" of the meter. This rating is done by moving the meter at various known velocities through still water in a reservoir, pond, or canal. Then knowing the velocity of the meter through the water and its readings, a rating curve or table of coefficients can be worked out.

41. Use of Floats. Very often a meter is not at hand, and a less accurate method must be employed. That most often used is by means of floats. These are of three kinds—*surface floats*,

subsurface floats, and *rod floats*. The best form is the rod float.

The *rod float* is a rod of wood, or a tube of tin, which is weighted at one end so that it will float in an upright position and as near to the bottom of the stream as practicable. The float is then placed in the stream at the desired point, and far enough up stream from the upper of two measured cross-sections so that it will acquire the same velocity as the water by the time it reaches such section. The time of its passage from the upper to the lower section is then observed and its velocity deduced therefrom. In this way observations are made for each of the vertical strips in which the stream section is divided. The average velocity of each strip is taken equal to that of the rod itself.

The *surface float* may be made of any convenient form which will be readily seen from the point of observation. Its use will give only the surface velocities of the several strips and not the desired average velocities. To get the average velocity, we may use the approximate formula,

$$\text{Average velocity} = .9 \times \text{surface velocity} \qquad \textbf{(35)}$$

whence the discharge of the several strips can be calculated as before. This method is not so accurate as the use of rod floats and is not to be recommended except for very rough determinations. It is much influenced by the wind, and observations should, if possible, be made on still days.

Sometimes a very rough determination is desired from one or two measurements of velocity. If the surface velocity is measured at a point where it is a maximum (near the center of the stream), then the average velocity for the entire stream may be taken at about $\frac{8}{10}$ of the measured velocity, although the exact value of this coefficient will vary between quite wide limits. The discharge then equals the total cross-section multiplied by the average velocity.

The *sub-surface float* consists of a submerged body a little heavier than water that is attached by means of a fine cord to a surface float of much smaller size. The sub-surface float can be adjusted to float at any desired depth. By setting it at mean depth the observed velocity will be approximately the average velocity of the vertical strip. The use of such floats is not looked upon with much confidence. Rod floats are much better.

REVIEW QUESTIONS

ON THE SUBJECT OF

WATER SUPPLY

PART I

1. How does the use of water vary month by month and day by day?

2. How do surface and ground waters compare generally as to quality?

3. What is a fair amount of water consumption per capita for various purposes?

4. If rain is falling at the rate of 4 inches per hour and the run-off is one-half as fast, what will be the flow in cubic feet per second from a drainage area of 10 square miles?

5. If the least annual run-off of a drainage area of 10 square miles be equal to 8 inches in depth, how many people will this provide for if the consumption averages 100 gallons per head per day, assuming there is storage capacity sufficient to utilize all the run-off for the year?

6. What storage capacity will be required in the above case if all the 8 inches runs into the reservoir in 5 months, leaving 7 months' demand to be met from the reservoir?

7. What conditions make it possible to secure artesian wells?

8. In what sort of material are we likely to find the most ground water available?

9. About what rate of consumption for fire purposes would be expected in a city of 25,000 inhabitants?

10. What causes the occurrence of springs?

11. What causes water to flow through the ground?

12. What are the most important uses of a public water supply?

13. What are the advantages and disadvantages of timber dams?

14. What considerations determine the location of lake and river intakes?

REVIEW QUESTIONS

ON THE SUBJECT OF

WATER SUPPLY

PART II

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1. Calculate the necessary thickness of a cast-iron pipe to carry a water pressure of 175 lb. per sq. in. pipe 12 in. in diameter.

2. If cast-iron pipe costs $30.00 per ton, what will be the cost of one mile of 8-in. pipe designed for a 250 ft. head?

3. Under what conditions are masonry conduits the most suitable forms of conduit for carrying water?

4. Compare the masonry conduit with iron pipe in regard to cost, durability, and the conditions under which they are the best form of construction.

5. When may conduits of vitrified clay pipe be used to advantage?

6. What is the function of a distributing reservoir?

7. Under what conditions is it desirable to employ reservoirs of earth; of masonry; of steel in the form of tanks or towers?

8. What capacity must a tank have to store water sufficient for one hour's fire use at a reasonable maximum rate in a town of 8,000 inhabitants?

9. What is the use of puddle in reservoir walls?

10. What precautions are to be observed in the construction of reservoir embankments?

11. What are the advantages of covered reservoirs?

12. Determine the thickness of a standpipe at points 10 feet apart from the top downward whose dimensions are: height 120 ft.; diameter 18 ft.

13. What is the uplift on the leeward side of this pipe at the bottom, due to wind pressure of 50 lb. per sq. ft.?

REVIEW QUESTIONS

ON THE SUBJECT OF

WATER SUPPLY

PART III

1. Discuss the care of household filters.

2. What are the various methods of sterilizing water for drinking purposes?

3. Discuss the various methods of water purification.

4. Discuss the object in the purification of public water supplies.

5. Discuss methods of operation of settling basins.

6. What is the theory of filter action?

7. Discuss the general construction of slow filter beds.

8. Discuss methods of cleaning filters.

9. What preliminary treatment of water is necessary for slow sand filtration?

10. What are the essential elements in a complete rapid filter plant?

11. What is the purpose of aeration and how is it accomplished?

12. Discuss the kind of sand for, and construction of, sand beds.

13. Discuss the results of varying pressure heads on filters.

14. Discuss the action of sedimentation.

15. What are the common substances used as coagulants?

REVIEW QUESTIONS

ON THE SUBJECT OF

SEWERS AND DRAINS

PART I

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1. Explain what is meant by the *separate* system of sewerage, and name its advantages and disadvantages as compared with the *combined* system.

2. Name the different materials used for sewers, and state the conditions to which each kind of material is adapted.

3. What is a *subdrain*, and when and why should subdrains be used?

4. What should be the minimum size of separate sanitary sewers and why? Of storm sewers, and why?

5. Why is the egg shape of sewer sometimes used? For what kind of sewers is it advantageous?

6. What nations of antiquity built the first known sewers? When did the scientific design and construction of sewers become general?

7. Give the principal objections to the use of cesspools.

8. What difficulties are encountered in designing and constructing the junctions of large sewers, and what designs are generally adopted for junction chambers?

9. What are the general facts as to the fluctuations in the flow of sanitary sewage at different hours of the day and night?

10. How would you determine the probable flow of sanitary sewage per capita per day, for any particular sewer system? Between what limits would the per capita per day flow probably lie in different systems?

11. How large capacities should the different kinds of separate sanitary sewers have as compared with the average flow of sewage in them?

REVIEW QUESTIONS

ON THE SUBJECT OF

SEWERS AND DRAINS

PART II

---

1. At 15 cents per cubic yard, what will be the cost of a drainage ditch 1 mile long, 6 feet average depth, and 20 feet average width?

2. What is a *trap*, what are the purposes of traps, and where should they be used? What kind of traps are best?

3. What size and kind of pipe should be used for house sewers, and at what grades should they be laid?

4. What average width of drainage ditch, carrying 5 feet depth of water, will be required to take the drainage of 50,000 acres of land, the grade being 5.3 feet per mile?

5. Estimate the cost, complete, under average conditions, of a 3-ring circular brick sewer, 9 feet in diameter, the average depth to the invert being 16 feet, and the length being 4,900 feet, with 12 manholes.

6. Explain the proper arrangement for the ventilation of a system of plumbing. Why is it necessary to connect the high point of each trap on the sewer side to the main ventilation pipe?

7. State in your own language the principal advantages of tile land drains; of open-ditch drains. Compare tile drains and open ditches for draining land.

8. What size of sewer subdrain, laid to a 0.35 per cent grade, will be required for outlet for 26,000 feet of tributary sub-drains, under ordinary soil and ground-water conditions?

9. What is a *soil pipe;* of what size and material should it be made, and how high should it be extended?

10. Explain how and why the composition of sewage varies even in the same sewer.

REVIEW QUESTIONS

ON THE SUBJECT OF

HYDRAULICS

1. What will be the exact weight of one U. S. gallon of distilled water at a temperature of 160° F.?

2. How much higher will a water barometer stand at a place 500 feet above sea level than it will when 4,000 feet above sea level?

3. Assuming that it is practicable to lift water by suction a distance equal to three-fourths the theoretical height as shown by the water barometer, how high is a suction lift practicable at a place located 6,000 feet above sea level?

4. What will be the pressure per sq. inch on the bottom or side of a vessel containing water, at a point 20 feet below the water surface?

5. In Fig. 2, how heavy a weight W will be supported by a weight P equal to 100 pounds, the area of W being 20 sq. in. and that of P being 2 sq. in.?

6. What is the total pressure on the side of a vessel 10 in. wide and filled with water to a depth of 8 inches?

7. What is the pressure on the face of a plate 10×10 in. in area, inclined at an angle of 10° from the vertical, and submerged so that the center of the plate is 30 in. below the surface?

8. What are the horizontal and vertical components of the pressure in Question 7?

9. What is the stress per sq. in. in a pipe 10 in. internal diameter and $\frac{1}{2}$ in. thick, under a pressure head of water of 100 lb. per sq. inch?

10. If the safe stress on cast iron is 5,000 lb. per sq. in., what is the necessary thickness of a cast-iron pipe 12 in. in diameter to resist a bursting pressure due to a head of water of 150 feet?

INDEX

The page numbers of this volume will be found at the bottom of the pages; the numbers at the top refer only to the section.

Note.—For page numbers see foot of pages.

Note.—For page numbers see foot of pages.

Note.—For page numbers see foot of pages.

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